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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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RAINFALL CHARACTERISTICS AND THEIR RELATION TO SOILS AND RUN-OFF*

By C. S. Jarvis,† M. Am. Soc. C. E.

Synopsis

Investigators of projects involving either flood control or the development of water resources frankly recognize the need for additional records of primary rainfall observations, and also for a fuller use of those already collected and published.

Ever-increasing and voluminous reports appear from time to time dealing with meteorology, hydrography and soil moisture; with the development, surveys, and classification of soils; with erosion and its control; with the varying yields of water-sheds; with the volume of floods, and their probable frequency of occurrence. All these must be rigorously condensed in some manner before they can be used to the best advantage.

The purpose of this paper is to present summarized information regarding precipitation and its occurrence in various countries and latitudes; its relation to soils and run-off; the vegetative, topographic, and physiographic features of the water-sheds; and the resultant influence on designs of drainage structures and channels.

Form of Data

Precipitation records for the principal stations throughout the world have been assembled in compact form in Table 1 (Appendix I), with both the average habits and the observed range of variation clearly represented. The references from which these data were obtained are given in Appendix II. Wherever snow or hail is included in the observations, it is reduced to equivalent rainfall.

Note.—Written discussion on this paper will be closed in **May, 1930**, *Proceedings*.

* Revision and extension of a thesis submitted for the degree of Ph. D., at American University, Washington, D. C., 1927; written with co-operation of U. S. Bureau of Public Roads.

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FIG. 1.—KEY MAP OF WORLD SHOWING SEQUENCE OF STATION NUMBERS FOR PRECIPITATION RECORDS—EASTERN HEMISPHERE.

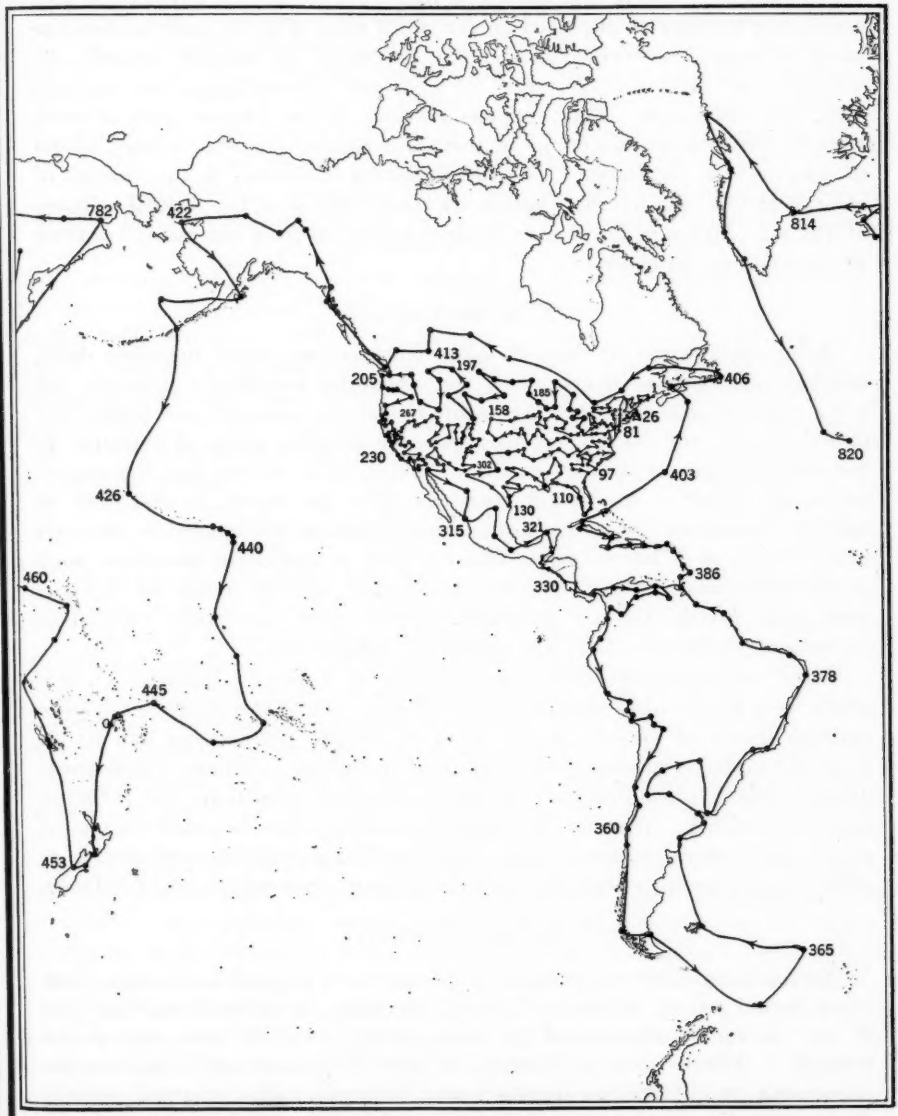

FIG. 1.—KEY MAP OF WORLD SHOWING SEQUENCE OF STATION NUMBERS FOR PRECIPITATION RECORDS—WESTERN HEMISPHERE AND PACIFIC OCEAN.

The selection, arrangement, and sequence of data were governed by such considerations as length and quality of records, directness in routing the path connecting stations as displayed on the world map (Fig. 1), and the development of progressive variations in: (1) Latitude; (2) distance inland; (3) altitude; (4) wind direction; and (5) character of contiguous land surface, soils, and vegetation. Thus, various portions of the adopted path between stations represent cross-sections of mountain ranges, plateaus, valleys, island groups, or even continents. The corresponding variations in the amount of rainfall and its seasonal distribution are readily traced in the vertical columns of Table 1 (Appendix I), where a gradual transition from one rainfall pattern to another may be observed.

Climatic Variation

After the first two or three decades of continuous record the mean daily, monthly, and annual temperature, the humidity, barometric pressure, the wind velocity and direction, the rainfall, and the snowfall are fairly well established for any station, together with the probable range of variation in these phenomena for similar periods. Further, it is evident that the annual means are subject to the least fluctuations. For that reason, in this study of rainfall characteristics the mean annual precipitation was adopted as the basis of reckoning, or as 100 per cent. Other critical or significant quantities, such as the maximum observed monthly precipitation and the mean for the corresponding month, also the maximum for 24 hours, have been reduced to percentages on the same basis for convenient comparison.

Inspection of the ten percentage columns in Table 1 (Appendix I), discloses more than a mere semblance of harmony. Beside the somewhat definite patterns traced by various groups, there are orderly progressions and trends from one group to another, with variations in latitude, altitude, temperature, distance inland, prevailing wind direction, and other conditions that influence relative humidity. Whether the tentative classifications be based on one or all of those factors, the great mass of divergent data conforms more readily to a few simple patterns of behavior than the primary observations would indicate.

Climatic Stability

No marked progressive changes of climate or of rainfall habits have been noted during several centuries of record; therefore, by determining what part of the pattern is represented by fragmentary, detached, and short-period records, a definite clue or forecast is provided concerning meteorological phenomena to be expected during longer intervals. The observed periodic changes usually complete their fluctuations within forty years, or thereabouts, according to authoritative opinion among meteorologists; but some stations require only twenty, whereas others need more than seventy, years for that purpose.

Determinate Limits

While it has long been recognized that there may be somewhat definite limits of rainfall intensity, frequency, duration, and variation applicable to given localities, an appreciation of the limited scope of human knowledge compared with the broad fields awaiting exploration deters the expression of opinions among those most competent to speak. Probably approximations to the limiting rates of precipitation for various periods and localities, as well as a measure of the resultant excessive run-off or surface drainage known as floods, would be more clearly indicated by arranging, co-ordinating, and interpreting primary observed data more effectively.

Such natural phenomena may be regarded as successive integrations involving many variable complex influences, some of which are interrelated while others are independent. Fortunately, meteorological and hydrographic records afford means for occasional evaluation of the quantities delivered, and further determinations may result from such simple processes as proportion and interpolation. In this manner solutions of practical problems reported herewith were undertaken during the assembling and tabulation of the accompanying basic data.

New Views of Old Problems

The need for flood control has come into prominence with every extensive inundation of highly developed valleys. Following the recent disasters sustained in widely separated districts of the United States, where flood-protection systems had been installed and immunity from serious damage had been supposedly secured, the public conscience has been awakened to the National responsibility in this vital problem and its solution. The resulting legislative enactments of 1928 which aimed to combine the most modern methods with the most positive means for effective flood control to promote the general welfare, are as noteworthy as were the catastrophes which prompted the legislation.

In carrying out flood-control programs recently authorized by Congress, engineers need to utilize every available means for extending their knowledge of rainfall characteristics, run-off rates corresponding thereto, and related phenomena in the vicinity of selected projects. Additional meteorological and hydrographic stations will be required to supplement those already established for the collection of primary data; but it is equally important to devise methods for assembling, co-ordinating, and utilizing the detached and apparently unrelated fragments of such records. If this can be accomplished, the effective periods of observation will be thereby extended far beyond their present limits, and the designs for retarding basins and waterways may embody proper consideration for both economy and safety.

If the source of supply and also the most frequented routing or paths of storms are known for a given area, and some fragmentary data are

available to disclose any part of the rainfall pattern, then the information in Table 1 (Appendix I), should furnish something of an index for completing the other parts of the record by comparison with available data pertaining to stations similarly situated and drawing on the same supply. Apparently, the characteristic percentage relations hold fairly consistently in such cases, regardless of the variation in rainfall traceable to the physical configuration of the land surface, except in arid regions.

Outstanding examples to illustrate that principle are furnished by the percentage columns for New England stations, others in Germany, and such adjacent but quantitatively varying stations, as Oviedo and Bilboa, in Spain, Nos. 683 and 684; Ben Nevis and Fort William, in Scotland, if adjusted to the same period of record, Nos. 717 and 718; and Tiflis and Lenkoran, in Trans-Caucasia, Nos. 762 and 763.

Both length and reliability of records, as well as completeness of detailed information and its representative character for either groups, areas, or the range of their habits, were sought among stations included in Table 1 (Appendix I).

Interrelation Between Meteorological and Hydrological Phenomena

An adequate understanding of the cause-and-effect relation between rainfall and run-off must likewise include the range of conditions which may influence the rate, sequence, and continuity of the various natural processes involved in the rain-producing cycle. Such conditions may include solar radiation, atmospheric circulation and convection, evaporation, expansion, cooling, condensation, deposition, percolation, stream flow, and the accretions or losses sustained en route.

Often arrested or interrupted processes are observed in the meteorological cycle, such as extended cloudy periods without rainfall, or the more advanced stage in which a downpour actually begins in the upper atmosphere, but is dissipated in the lower strata before reaching the earth's surface, producing the familiar cloud-drapery appearance above desert regions. Likewise, the detention and regulation effected by the soil and vegetation during moderate showers may interrupt the run-off and reduce it to a negligible quantity. Under ideal conditions practically all the rainfall percolates into the voids of the soil, and emerges therefrom at a gradually decreasing rate until new accessions of moisture are available.

Maximum river stages usually result from the coincidence of either intense rates or prolonged periods of precipitation or of thawing, together with low absorptive, interceptive, and retentive powers of the soil and its cover.

Water-Shed Characteristics

Types, structure, color, and stages of development of soils and the varieties of plant growth sustained thereon, provide a supplemental record awaiting

interpretation as to rainfall and run-off habits, whether they be irregular and violent or steady, gentle, and dependable. Thus, the 143 sq. miles of drainage area along Queen Creek, in Central Arizona, with average annual precipitation of 15 in., produced a high flood volume of 9 000 cu. ft. per sec., but maintained an average flow of only 15 cu. ft. per sec. throughout the year, representing a yield of 10% from the water-shed; by comparison, an equal area along Cedar Creek, in Northwestern Washington, with an average annual rainfall of 120 in., attained an observed maximum flow of 3 600 sec-ft., a minimum of 294 cu. ft. per sec., and an average discharge of 1 089 cu. ft. per sec., a yield representing nearly 90% of the total observed precipitation.

Both the soils and the vegetation that abound in those two regions emphasize the contrast between the steady, gentle, dependable rainfall of the Pacific Northwest as opposed to the sudden, irregular, brief, torrential downpours that are so characteristic of desert regions. The soft, mossy banks, the tender foliage, and the sodded slopes are identified with precipitation which develops almost imperceptibly from fog or mist; while the spike foliage, thorny stems, and other resistant equipment of desert growth reflect the struggle for existence.

There are known physical limitations as to the moisture capacity of the atmosphere for given temperatures and pressures, and there may be just as definite limits for barometric pressure gradients and the wind velocities induced thereby; likewise for storm movements and areas affected, rates of evaporation and condensation, heating and cooling, also freezing and thawing, for designated seasons and localities.

Limitations Governing Atmospheric Moisture

The high transparency of clear, dry air allows the transmission of nearly 90% of the normal solar radiation under most favorable conditions. The losses due to reflection, absorption, and conduction are all increased by the presence of foreign substances, including water vapor, so that the radiant energy reaching the earth is reduced accordingly. After condensation has proceeded far enough to form dense cloud masses, practically the entire solar radiation is intercepted.

An average measure of the sun's radiant energy that reaches the outer atmosphere may be expressed as 2 calories per sq. cm. per min., or $\frac{1}{8}$ h.p. per sq. ft. of normal surface continuously during sunlight hours. If transmitted to the earth without loss, this energy would be capable of evaporating 1 in. depth of water at 72° Fahr. during 12 hours near the equator, or from two-thirds to one-half this quantity in mid-latitudes. However, the actual toll of energy taken by the atmosphere during average fair weather conditions is nearly 50%; and during cloudy periods it becomes 90%, or more.

Taking into account the varying opportunities for evaporation as determined by the prevailing temperatures, the inclination of the sun's rays, the

TABLE 2.—Record of Evaporation Rates.

(All depths in inches.)

Station	Years of record.	Maximum.			Mean.	Annual.			Remarks.
		Day.	Mean month.	Month.	April to September.	Minimum.	Mean.	Maximum.	
Gardiner, Me	3	.....	5.0	5.8	22.8	..	26	..	.....
Boston, Mass	15	.....	6.0	7 4	28.6	31	39	48	.....
Rochester, N. Y	25	.....	4.7	6.9	....	26	31	39	.....
Washington, D. C	3	.....	...	7.0	32.2	..	..	..	.....
Chapel Hill, N. C	4	.....	5 3	7.5	....	..	40	..	.....
Charleston, S. C	20	.....	5.7	8.6	....	30	47	66	Reservoir.
Charleston, S. C	19	.....	10.4	....	....	..	88	..	Land pan.
Birmingham, Ala	..	.....	7.5	....	38.3	..	51	..	.....
California, Ohio	..	.....	7.2	....	35.5	..	46	..	.....
Cleveland, Ohio	6	...	...	....	24.6	..	..	..	.....
Detroit, Mich	4	.....	...	....	32.1	..	..	..	.....
Thunder Bay, Mich	4	.....	...	....	30.2	..	..	..	.....
Milwaukee, Wis	5	.....	...	....	26.9	..	..	..	.....
Lake of the Woods, Minn	3	.....	3.5	....	20.7	..	..	..	.....
Columbia, Mo	2	.....	...	7.5	31.2	..	50	..	.....
Crowley, La	7	.....	6.2	....	32.3	..	..	..	.....
Laredo, Tex	1	.....	...	13.7	68.4	..	..	..	.....
San Antonio, Tex	11	.....	9.6	....	46.0	..	..	..	.....
Austin, Tex	2	.....	...	11.0	47.3	..	68	..	.....
Amarillo, Tex	11	.....	10.4	....	52.7	.	..	..	.....
Big Springs, Tex	3	.....	12 7	....	59.0	..	..	..	.....
Dalhart, Tex	10	.....	10 6	....	55.4	..	..	..	.....
Lawton, Okla	3	.....	9.0	....	45.6	..	..	..	.....
Woodward, Okla	4	.....	10.8	....	49.5	..	..	..	.....
Garden City, Kans	10	.....	10 9	...	52.8	..	..	..	.....
Hays, Kans	11	.....	10.0	....	47.9	..	..	..	.....
North Platte, Nebr	11	.....	9.1	....	43.6	..	54±	..	.....
Lincoln, Nebr	11	.....	....	....	34.8	..	..	..	.....
Newell, S. Dak	16	.....	8 1	....	36.6	..	..	..	.....
Edgeley, N. Dak	11	.....	6.4	....	29.6	..	..	..	.....
Mandan, N. Dak	5	.....	7 4	....	33.3	..	..	..	.....
University, N. Dak	16	....	5.7	7 1	26.2	..	..	..	.....
Huntley, Mont	13	.....	7.5	....	32.8	..	..	..	.....
Moccasin, Mont	9	...	7.1	....	32.8	..	..	..	.....
Archer, Wyo	5	....	7.9	....	36.3	..	..	..	.....
Laramie, Wyo	6	...	....	....	39.4	..	..	..	.....
Fort Collins, Colo	16	.....	....	....	29.3	....	....	....	.....
Akron, Colo	10	.....	9.1	....	42.7	....	....	....	.....
Grand Valley, Colo	1	.....	....	....	36.5	....	....	....	.....
Rocky Ford, Colo	1	...	....	....	61.6	....	....	....	.....
Tucumcari, N. Mex	6	.....	10.9	13.8	55.4	....	....	....	.....
Santa Fe, N. Mex	2	.....	....	11.9	52.5	....	....	....	.....
Elephant Butte, N. Mex	7	.....	14 3	...	....	90	102	110	.....
Elephant Butte, N. Mex	2	.	....	16.9	75.8	...	....	....	.....
Elephant Butte, N. Mex	..	.....	13.5	....	....	....	87	...	.....
Carlsbad, N. Mex	4	....	....	12.4	69.4	...	107	....	In city.
Carlsbad, N. Mex	4	.....	....	11.1	....	....	94	....	In alfalfa field.
Lake Avalon, N. Mex	10	.....	10.0	12.9	....	66	73	83	.....
Las Cruces, N. Mex	3	.....	....	....	40 1	....	....	....	.....
Holbrook, Ariz	5	.....	....	8.6	47.0	....	....	....	.....
Roosevelt Reservoir, Ariz	7	.....	13.4	14.8	67.0	82	87	95	.....
Granite Reef, Ariz	1	.....	....	12.7	....	....	98	....	Tank floating in river.
Granite Reef, Ariz	1	.....	....	14.3	....	....	115	....	Tank on ground
Mesa, Ariz	2	.....	12 2	....	56.2	....	91	....	.....
Tucson, Ariz	3	....	....	....	54.2	....	....	....	.....
Yuma Desert, Ariz	3	.....	17.5	....	....	114	123	136	.....
Yuma, Ariz	9	.....	10.6	....	55.1	....	96	....	.....
Yuma, Ariz	5	.....	10.4	....	....	70	76	88	Alfalfa field.
Nephi, Utah	10	....	9 5	....	44.6	...	....	....	.....
Spanish Fork, Utah	5	.....	9.6	....	38.8	....	....	....	.....
Fort Douglas, Utah	3	....	....	....	30.7	....	....	....	.....
Aberdeen, Idaho	6	.....	9 6	...	42.1	....	....	....	.....
Arrowrock Idaho	8	....	....	10.2	42.0	....	....	....	.....
Deer Flat, Idaho	8	...	8.0	....	38.1	....	96	....	.....
Minidoka, Idaho	..	.....	....	15.0	....	....	96	....	.....
Walla Walla, Wash	2	.....	....	9.5	38.0	....	....	....	.....

TABLE 2.—(*Continued*).

(All depths in inches.)

Station.	Years of record.	Maximum.			Mean.	Annual.			Remarks.
		Day.	Mean month.	Month.	April to September.	Minimum.	Mean.	Maximum.	
North Yakima, Wash.	..	.....	....	10.7		....	68	....	.....
Lake Kachess, Wash.	8	.....	....	6.1	23±	....	33	...	.....
Klamath, Ore.	4	.....	8.5	9.2	....	...	51	....	.....
Hermiston, Ore.	6	.....	8.5	11.1	36.6	....	....	....	.....
Hermiston, Ore.	..	.....	8.4	....	....	....	44	....	.....
Moro, Ore.	7	.....	9.3	....	41.0	....	...	....	.....
Fallon, Nev.	16	.....	10.7	....	50.9	61	64	73	.....
Fallon, Nev.	..	.....	....	9.9	....	....	54	....	Tank floating in canal.
Lake Tahoe, Calif.	7	....	4.9	6 5	....	20.5	30	40.1	.....
Lake Tahoe, Calif.	8	....	5.9	....	....	....	40	....	.....
Biggs, Calif	4	.....	9.6	....	....	....	44	....	.....
Sweetwater, Calif.	7	.....	....	9.0	....	49	54	59	.....
Cuyamaca, Calif.	9	.....	....	....	....	45	57	85	.....
Fresno, Calif	..	.....	....	....	....	....	66	....	.....
Los Angeles, Calif.	..	.....	....	....	...	....	37	....	.....
San Diego, Calif	..	...	....	....	....	....	38	....	.....
Mammoth, Calif	..	.....	....	18.0	....	....	126	....	.....
Owens Lake, Calif.	..	....	....	....	....	....	61	....	.....
Christiansted, Virgin Islands.	8	.....	8 2	....	45.2	74	78	84	.....
Assuan, Egypt	3	.....	....	....	....	....	107	....	.....
Bombay, India	..	.....	....	....	....	....	96	....	.....
Van Wijks Vlei, South Africa	14	.....	....	....	....	....	67	....	.....
Kimberley, South Africa.	8	.....	....	....	....	79	89	93	Vaal River.
Kimberley, South Africa.	8	.....		....	....	....	55	....	Deep reservoir.
Bloemfontein, South Africa	..	.....	....	....	....	66	....	82	Exposed to wind.
Bloemfontein, South Africa	..	.....	....	....	....	48	....	58	Sheltered from wind.
Johannesburg, South Africa	..	.....	....	....	....	....	75	....	.....
Capetown, South Africa.	..	.	....	...	....	...	79	....	.....
Port Elizabeth, South Africa.	..	.....	....	...	....	32	....	46	.....
Alice Springs, Australia.	24	0.88	12.5	15.5	....	34	96	108	.....
Adelaide, Australia	47	0.62	9.0	11.2	....	47	54	61	.....
Eucla, Australia	6	0.52	7.0	8 3	....	53	58	61	.....
Brisbane, Australia.	3	0.48	....	8 2	....	49	51	55	.....
Blackall, Australia.	4	0.70	11.5	13.0	....	77	83	92	.....
Boulia, Australia.	3	0 91	....	16.7	....	....	111	125	.....
Murrumbidgee, Australia.	5	...	....	....	....	29	37	45	.....

soil moisture content, the degree of cloudiness, the vegetation and foliage, the wind movements, and the atmospheric humidity, it is estimated that the equivalent of 0.1-in. depth of water over the entire surface of the globe becomes vapor each day, on an average. The upper limit of evaporation rates attainable from free water surfaces may equal 1 in. daily. Illustrative of local variations, the recorded annual evaporation from a tank on the ground at Indio, in the Imperial Valley, California, was 119 in.; from a tank 10 ft. above the ground, 200 in.; from the saline waters of the Salton Sea itself, 72 in.; from a tank 2 ft. above the surface of Salton Sea, 106 in.; from a tank 40 ft. above the water, 140 in.; from a tank on the desert, 165 in.; and from one that was elevated 40 ft. above the desert, 195 in.

Representative evaporation rates for several important stations are shown in Table 2. Main references to this table may be found in Appendix III.

Evaporation Compared with Precipitation

Comparison of the corresponding columns in Table 1 (Appendix I) and Table 2 indicates the normal relationship between evaporation and precipitation phenomena, especially as regards maximum observed rates. The evaporation process is more or less continuous, while rainfall is usually periodic; the one is governed by atmospheric circulation and temperatures affecting surfaces of known or readily ascertainable areas, while the other depends on condensation of cloud volumes at considerable heights, in air strata which may be moving in different or opposite directions from those below.

TABLE 3.—Moisture Capacity of Atmosphere at Various Temperatures.*

Temperature, in degrees Fahrenheit.	Weight, in Pounds, of 1 000 Cu. Ft. of Air.		Moisture Percentages by.		Equivalent Depth of Water, in Inches.	
	Dry.	Saturated.	Weight.	Volume.	Per 1 000 ft. of air column.	Summation for successive strata.
−45	95	95	0.01	0.01	0.001	0.001
−40	94	94	0.01	0.01	0.001	0.002
−35	93	93	0.01	0.02	0.002	0.004
−30	92	92	0.02	0.03	0.003	0.007
−25	91	91	0.02	0.04	0.004	0.011
−20	90	90	0.03	0.05	0.005	0.016
−15	89	89	0.04	0.06	0.006	0.022
−10	88	88	0.05	0.08	0.008	0.030
− 5	87	87	0.06	0.10	0.010	0.04
0	86	86	0.08	0.12	0.013	0.05
5	85	85	0.1	0.2	0.02	0.07
10	84	84	0.1	0.2	0.02	0.09
15	83	83	0.2	0.3	0.03	0.12
20	83	83	0.2	0.3	0.03	0.15
25	82	82	0.3	0.4	0.04	0.19
30	81	81	0.3	0.5	0.05	0.24
35	80	80	0.4	0.6	0.06	0.30
40	79	79	0.5	0.8	0.08	0.38
45	78	78	0.6	1.0	0 09	0.47
50	78	78	0.7	1.2	0.11	0.58
55	77	77	0.9	1.4	0.14	0.72
60	76	76	1.1	1.8	0.16	0.88
65	75	75	1.3	2.1	0.19	1.07
70	75	74	1.5	2.5	0.22	1.29
75	74	73	1.9	3.0	0.26	1.55
80	73	72	2.2	3.6	0.30	1.85
85	72	71	2.6	4.2	0.35	2.20
90	72	71	3.0	4.8	0.41	2.61
95	71	70	3.5	5.6	0.47	3.08
100	71	69	4.1	6.6	0.54	3.62
105	70	68	4.8	7.[illegible]	0 63	4.25
110	70	68	5.5	8.9	0.72	4.97
115	69	67	6.4	10.3	0.82	5.79
120	69	66	7.4	11.[illegible]	0.94	6.73
125	68	65	8.6	13.[illegible]	1.07	7.80

*These average values were computed for normal pressure at sea level and were based mainly on Smithsonian Physical Tables. The last column quantities represent moisture content subject to precipitation.

Except for those few stations of extremely high or low rainfall, there are corresponding values to be found for both monthly and yearly means and maxima; but the range of variation from normal is not so wide for evaporation, representing receipts of atmospheric moisture, as for precipitation, representing expenditures. The brief mid-day showers of the tropics exemplify the expenditure of an entire day's accumulation of atmospheric moisture in less than an hour. The maximum rates recorded in Table 1 (Appendix I) for 15-min. periods occasionally approach or exceed the greatest observed daily evaporation from free water surfaces. Many of the inequalities as well as the similarities displayed by those two phenomena are traceable to limitations governing atmospheric moisture capacity.

Under the extreme conditions immediately preceding a storm, with the temperature as high as 110° Fahr. at sea level, for example, and the air column closely approaching saturation, the moisture subject to precipitation is nearly 2.8 in. in the first mile above the base; 1.3 in. in the second mile; 0.6 in. in the third; and 0.3 in. above the 3-mile level, according to Table 3. For such a condition, a 5-in. depth of rainfall (Table 3) may be the attainable limit without renewal of the moisture supply by wind movements. Whether a multiple or only a fraction of that amount may be registered as precipitation is beyond present human knowledge unless the records of past performance may serve as a guide.

TABLE 4.—Comparisons of Computations.*

Station.	Season.	Moisture Capacity of Saturated Atmosphere According to Field Observations.					Corresponding Values from Table 3.†
		Observed Data.				Computed Data.	
		Latitude, in degrees.	Temperature, in Degrees Fahrenheit.		Height explored, in miles.	Moisture Capacity at Saturation.	Portion subject to precipitation, in inches.
			Maximum.	Minimum.		Depth, in inches.	
McMurdo Sound..	Summer	78 S.	18	— 49	3.7	0.20	0.14
Arctic Sea	Summer	77 N.	41	— 62	6.8	0.80	0.40
Pavlovsk.....	Summer	59 N.	52	— 54	8.7	1.16	0.64
	Winter	59 N.	18	— 69	7.4	0.28	0.14
England.....	Summer	52 N.	61	— 60	8.7	1.40	0.92
	Winter	52 N.	37	— 71	8.7	0.60	0.33
Canada	Summer	43 N.	68	— 74	6.8	1.84	1.20
	Winter	43 N.	30	— 80	9.3	0.60	0.24
St. Louis, Mo.....	Summer	38.6 N.	77	— 60	9.3	2.28	1.67
	Winter	38 6 N.	34	— 80	9.3	0.68	0.29
Atlantic Ocean...	Summer	20.7 N.	77	— 83	8.7	2.60	1.67
Victoria Nyanza..	Summer	0	73	—108	12.5	2.92	1.41

* In the Dictionary of Applied Physical Science, Vol. III, p. 61, is a chart displaying the data observed and derived by several carefully conducted series of investigations.

† As the portion of the moisture content at saturation which is subject to precipitation varies from nearly 50 to 75%, this is nearly the correct proportion of the total, shown in the preceding column.

Field observations recorded in Table 4 for atmospheric moisture capacities at widely separated stations have furnished a practical and satisfactory test for Table 3, whether applied to frigid, temperate, or torrid regions. In most instances both summer and winter conditions were observed.

Not all the moisture content of either the air or the soil is subject to removal by natural processes; a considerable percentage remains as residue continuously. The moisture that may be removed from the air column by precipitation during a storm is probably not more than one-half or three-fourths of the total capacity at saturation.

For various soil types the readily removable moisture may range from more than 90% for sand to less than 20% for heavy clay. The effective storage capacity and also the contribution to stream flow during dry seasons are reduced accordingly, and thus the opportunity for evaporation may be limited by what the soil will relinquish when the heat units and atmospheric capacity for moisture are both available.

It is evident that the normal rates of evaporation for various latitudes or for different climatological zones are such as would surcharge the atmosphere within a period of a month, or less, under any conceivable conditions, provided there is ample opportunity for evaporation from either free water surfaces or wet soil, or for transpiration from luxuriant vegetation and foliage. The surcharged atmosphere obtains relief either by increased temperature and moisture capacity, or by precipitation.

If proper allowance be made for the required interval of lag between the processes of evaporation and precipitation, together with the effects produced by progressive seasonal and local variations of temperature and relative humidity of the atmosphere, some of the characteristic rainfall habits may be explained, or, perhaps, they may be forecast. Likewise, the influence of barriers, whether they be mountains or masses of dense, cold atmosphere obstructing the path of moisture-laden air currents, can be partly evaluated.

The most dependable controls for such methods of analysis seem to be the long-period records kept by weather bureaus, supplemented by a consideration of the fundamental physical laws involved. Where only meager records are available, it seems advisable to attempt the expansion of fragmentary data in terms of what appears to be the attainable limiting conditions against which provision should be made in the design of waterways for a given locality. Table 1 (Appendix I) and Table 2 were compiled to furnish something of a background for interpreting the more detailed data which may be assembled for specific problems, in the same manner as previously used for hydrographic investigations.*

It appears that precipitation may vary either directly or inversely with evaporation, depending on local conditions which determine whether the moisture supply is steady or intermittent; whether the source is near-by or remote; whether the routes to be traversed by the winds are clear or obstructed,

* "Flood Flow Characteristics," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

both approaching and beyond the station in question; and whether the latent energy represented by the vapor content of the atmospheric column is capable of producing a violent, or only a moderate, disturbance upon release by condensation.

The Energy of Vaporization and Condensation.—The energy required to change 1 lb. of ice at 32° Fahr. to water amounts to 144 B. t. u., equivalent to the work of elevating that same weight 144 by 778 ft., or to a height of 21.2 miles. The same quantity of heat would raise 144 lb. of water from 60° Fahr. to 61°; or 1 lb. of water from 32 to nearly 176 degrees.

Evaporation of 1 lb. of water under average atmospheric pressure at sea level requires the expenditure of 1 069 B. t. u. at a temperature of 40°; 1 052 B. t. u. at 70°; or 1 036 B. t. u. at 100° Fahr. Thus, at ordinary temperatures and pressures, such as prevail while most of the process takes place, it requires approximately seven times as much energy to vaporize a given weight of water as to melt the same amount of ice; or if converted into units of work, it represents an expenditure that would elevate the same weight to a height of nearly 150 miles.

Because the relative weights of air and water vapor at equal temperatures are as 1.0 : 0.62, it is apparent that the presence of 3.0% by weight, or 4.8% by volume, of moisture to produce saturation at 90° Fahr., as shown by Table 3, will reduce the weight of the entire volume by 1.8% as long as condensation is delayed. A condition of marked instability, therefore, may result if opportunities for evaporation are alternately favorable and unfavorable. At 70° Fahr., 1%, and, at 110° Fahr., 3.4%, of variation in weight and consequent pressure may be due to the vapor content.

The condensation of 1 in. of rain releases the equivalent of 2 h.p.-hours of energy per sq. ft. of surface, or nearly 87 000 h.p.-hours per acre. If applied exclusively toward heating the first mile in height of air column above sea level, the resultant increase of temperature would be 30° Fahr., or more, depending on the density and humidity; and this thermal change would cause an expansion ranging from 6 to more than 10 per cent. The resulting buoyancy would be of the same order as that possessed by ice or logs floating in water.

Although it is improbable that all the latent energy released during a storm is devoted solely to heating a portion of the atmospheric column, it is definitely known that a large part of it induces energetic convection currents which continue until equilibrium is finally established or approached through ascent, expansion, cooling, intermixing, and condensation. Much of the heat is introduced into the soil by the absorption of rain water and thereby promotes bacterial activity, the release of soil fertility, and the rapid growth of vegetation, which is so noticeable after a storm, especially one that is prolonged and of only moderate intensity.

Time Element Involved in Evaporation and Condensation

Pure, dry air may transmit solar radiation with losses of less than 10% when the sun is near the zenith. Other highly transparent substances of

equivalent mass, such as a 34-ft. depth of clear, placid water, or a 14-ft. thickness of the best grade of glass, behave similarly. The amount of energy delivered from the sun at such times is capable of evaporating nearly $\frac{1}{12}$ in. in depth of water every hour, or of melting $\frac{1}{2}$ in. in thickness of ice, or of raising the temperature of either a 6-in. layer of dry soil, or of a 1 000-ft. stratum in the lower atmosphere fully 25° Fahr. Thus, a few consecutive hours of intense sunshine would render the tropics and even the temperate zones unsuitable for most forms of organic life, were it not for the moderating influences of moisture and wind.

Fully 12 hours of very bright sunshine, or the combined action of persistent wind and moderate temperature, are required to evaporate 1 in. in depth from a free water surface. Such a rate is rarely exceeded except in shallow pools or in places exposed to vigorous wind movements. Condensation of the same quantity ordinarily occupies several hours, but in extreme cases precipitation, the culmination of the process, occurs within a few minutes in greater amount. Virtually it is bulk delivery as opposed to the deliberate complexities of manufacturing.

The intimate knowledge and accurate interpretation of meteorological conditions which were so largely stimulated by the development of aviation have gradually taken definite form, and one of the new departures, that is, official weather forecasts a full week in advance, is well within if not beyond the experimental stage. Thereby the major storm movements and the general tendencies on large areas are foretold with nearly the same degree of accuracy as the more detailed local weather forecasts have attained for small areas.

Soils and Their Moisture Capacity

The products of rock-weathering ultimately become soils through changes wrought by micro-organisms with the assistance of climatic and biological forces. Prior to the bio-chemical alterations the decomposed or disintegrated material has not developed its character sufficiently to permit classification under soil types; it is regarded therefore as in the embryonic stage. As the biological activity extends only a few feet below the surface, and chemical, hydrolytic, thermal, and other disturbing influences are most pronounced in the same zone, the upper layer to a depth of 10 ft., or less, claims most of the attention of soil investigators.

Recent Conceptions Concerning Soils.—Since 1900 the foundation of a radically new conception of soil science has been established in Russia and developed in Europe and America, until it now occupies a major position commanding a broad field. Old ideas regarding the dominant influence of the parent ledge were abandoned after it was demonstrated that a given type of soil may develop from any of the most prominent varieties of rock magma, provided the environment is favorable. From this view has evolved the notion that soil is not a mere inert residue or product of decay, but a complex

entity, a creature of growth and culture with varying expressions of vitality. Its habits seem to reflect the influences of the temperature, rainfall, and drainage which governed its development, as well as that of lower forms of organic life which are so intimately associated with fertility and fruitfulness.

Carrying the simile still further, it has been aptly observed that the coarser granular particles constitute the framework or skeleton and determine the stability of the body structure. Again, the colloids combine the functions of tendons, muscles, linings, digestive system, and surplus tissue, for here the stores of essential minerals and other plant food lie quiescent until requisitioned by the circulatory system to supply the needs of vegetation.

There is a well-defined respiratory process by which oxygen is introduced and carbon dioxide removed; complexions of red, gray, brown, or black, disclose the degree of oxidation and hydration, and also the presence of iron or other coloring pigments that have survived the process of leaching. Here, as in organisms, impaired drainage facilities may produce a pronounced toxic condition. The living roots readily resist the soil digestive process, but the dead wood soon succumbs to the yeasts, moulds, and bacteria swarming within the colloids. Finally, soil degenerates under improper treatment and responds so gratefully to regular care and understanding that it seems like an expression of life and health, rather than of decay.*

Soil Characteristics

A comparison of the desirable qualities or characteristics of soils brings forth striking contrasts. For example, the presence of considerable organic matter is objectionable in soil for structural foundation and highway purposes; and it is either beneficial or vitally essential in the case of soil for agricultural use and stream regulation. For example, the presence of considerable organic matter is objectionable in the first case, and either beneficial or vitally essential in the second. On the other hand, high percentages of coarse, inert, granular fragments of rock are desirable for sustaining either structures or traffic and also for regulating run-off, inasmuch as stability and ready drainage facilities are among the prime requisites. These considerations, however, do not govern in farming operations; there, the sieve-like structure of coarse, gravelly, or sandy soil has become almost a synonym for sterility or lean production.

Where the only available sub-grade material is a fertile soil, it requires some sort of treatment to render it suitable for sustaining either structures or traffic. Dragging, rolling, or other manipulation while the soil is wet may afford improved stability and resistance to accessions of moisture. Both the results and the methods used are contrary to recommended practice in agriculture or in the maintenance of water-sheds to regulate run-off, for in such

* "Soils and Civilization," by Milton Whitney, Chf. of the Bureau of Soils, U. S. Dept. of Agriculture, p. 25, "The Soil as a Living Thing."

activities it is considered that the permeability, the normal capillary structure, and the organic life of the soil should be maintained and not destroyed. The readiness with which an overturned bucket of water is absorbed in a meadow or lawn, in contrast with its behavior on a well-trodden pathway where saturation is long delayed and where there is never danger of miring, illustrates the difference between productive soils under a cover of vegetation and the same materials after puddling and compression, erosion and baking, have rendered them sterile and inert.

It is an unusually close-textured soil that cannot absorb and temporarily hold 1 in. of rainfall for each foot in depth of the soil deposit, provided no previous saturation, freezing, baking, or mechanical compaction has occurred to seal the inlets. In the deep soils traversed by irrigation systems of the West, it is customary to sustain a loss of 10 to 20 in. of water daily over the entire wetted perimeter of earth canals, regularly throughout the season. In humid regions only unusual soil structure, such as occurs in coarse sandy or glacial deposits, would be capable of comparable behavior; yet in lesser degree all soils of close texture may function similarly, especially if covered with dense vegetation.

For ordinary rates of rainfall the toll taken by vegetation and soil to prevent or regulate run-off is much greater than the capacity of detention basins usually designed and provided by works of Man. The flood volume recorded in Lake Champlain within a week following the unusually severe storms of early November, 1927, was not more than one-half the actual precipitation as estimated by several independent investigators, including official observers. For restricted areas near the head-waters, nearly 90% appeared as run-off, but this may have been further depleted on its way to distant outlets.

Soil Classifications.—Soils are sometimes classified on a chemical basis, as silicate, carbonate, sulfate, or ferruginous deposits, each with suitable subdivisions. Another system utilizes the manner of deposition, as residual, transported, alluvial, colluvial, aeolian, or glacial, with subdivisions according to parent sources, as quartzite, granite, dolomite, or basalt.

The Russian scientist, Glinka, has proposed a general classification based on the moisture conditions under which the types were developed, whether excessive, optimum, average, moderate, or insufficient; with sub-classes such as tundra, podsol, chernozem, prairie, gray desert, laterite, and red loam, each identified by color, position, texture, composition, and details regarding the formation generally.

The United States Department of Agriculture, through the Bureau of Soils, has developed what appears to be the most logical and comprehensive classification yet devised, incorporating the outstanding features of all other important systems, each in its natural position. The primary category is based on the presence or absence of calcium carbonate accumulations; the second on the degree of oxidation, stage of leaching, formation of sesqui-

oxides, and the differences in organic content. The third category identifies the soil type, such as laterite, podsol, or chernozem; the fourth has to do with the "soil profile", that is whether or not the characteristic changes have developed at the various "horizons", or depths below the ground surface, from top-soil to sub-soil.

Further subdivision deals with geographical regions and soil provinces, as Piedmont, Upland Valley, or Coastal; the locality where a given type was first surveyed and classified, as the Upshur, Miami, or Volusian series; and, finally, the texture and composition are described, as gravelly loam, light silty clay, or heavy gumbo.

The results already achieved in the United States as well as abroad justify the belief that this system of classification may be adopted generally. Every major and minor series or type falls into its proper place in that system after a thorough field study and occasional laboratory tests have been completed.

Utility of Soil Surveys and Classification.—The obvious utility of soil analysis, research, surveys, and classification is to extend the horizon of knowledge and to make the best use of what has been gathered heretofore. Wherever a soil type is identified an intelligent judgment may be formed regarding its genesis and development, the climatic, rainfall, and drainage habits that prevailed, and the vegetation sustained during the progressive stages of its growth. In the brief period since soil science has been recognized adequately, it has rendered valuable service to agriculture in all its phases, to commerce, to the manufacturing industry, to water supply, to flood protection, and to the most extensive and important highway development yet undertaken.

What promises to be one of the most notable steps forward was inaugurated by the U. S. Bureau of Public Roads as sub-grade investigations. The work has been carried on in co-operation with various State highway departments and colleges. The net result thus far has been to narrow the problem down to the identification and treatment or the elimination of harmful elements. Inasmuch as the ordinary failures of pavements that are traceable in any manner to unstable sub-grades represent but a small proportion of the total mileage, attention has been concentrated on these sections. Further applications for soil science are yet to be realized in the broad fields of structural and foundation design, drainage, control of surface run-off and underground water supplies, crop production without serious depletion of fertility, and all lines of endeavor that depend on soil materials or properties.

Illustrative of the clues to soil characteristics which have long been familiar to even casual observers, a rank growth or a precarious one of either arrow-weeds or bud-sage, salt-grass or blue-grass, cactus or willows, may publish more information concerning the chemical analysis, texture, water supply, and drainage habits of the land than could be conveyed on several pages. If the principal vegetation is scrub cedar of the uplands, swamp cedar, or the giant redwood, radically different conditions are known to govern.

Classification of Precipitation Patterns

The time and labor of assembling and reducing the summarized precipitation characteristics presented in Table 1 (Appendix I) may prove to be meager as compared with what will be required to develop an adequate appraisal and interpretation of the data contained therein. It will probably remain for the discussions to bring out the practical relations and applications to a wide variety of problems.

A tentative classification is presented in Table 5, in which the first or primary basis of division depends on climatological zones, approximately defined by parallels of latitude. The irregularities introduced by such influences as ocean currents and prevailing winds, as illustrated in Siberia, Labrador, and the Scandinavian Peninsula, for example, seem to be fully as significant and important as the similarities displayed by other groups of stations. The second and third categories have to do with distances inland and altitude, respectively; and, finally, the representative stations to show both the range of habits and also the average behavior for specified conditions, are arranged according to the amount of either annual or monthly rainfall. Other selections might be made to display somewhat greater or less concordance than appears in Table 5, wherein both length and completeness of record were taken into account.

Outstanding features of Table 1 (Appendix I), and Table 5 are the manifestations of progressive increase of moisture capacity with temperature. It is notable that the greatest observed intensity of rainfall occurs in the sub-tropical, and not in the equatorial regions. Evidently, this is not due to lack of atmospheric moisture, but to the fact that some of the conditions necessary to the violent relinquishing of moisture are not to be encountered near the equator. For example, the polar fronts and cold air currents and icebergs are all unheard of in lower latitudes for the same identical reasons. Special situations might be encountered, however, in which the condensing medium would be supplied from the sub-zero temperature zones in the upper atmosphere, if it were not for the regular daily relief afforded by the mid-day showers of the tropics; and it is conceivable that rates of precipitation might occur during a brief period as intense as have been registered elsewhere.

The tabulations plainly indicate that there are fairly definite limiting conditions and rates of rainfall appurtenant to the various climatological zones. There are typical rainfall patterns displayed in the percentage columns of Table 1 (Appendix I), which may be utilized for combining fragmentary records, or for supplying missing data under proper controls.

Application to Practical Problems

In the solution of practical problems, the greatest needs are for more basic data and for a better means of interpreting those already available.

Inquiries as to what may be expected in the way of rainfall and run-off habits in a given locality are too often answered by reference to voluminous

TABLE 5.—Representative Precipitation Stations and Characteristics, from Table 1 (Appendix I).

Record of Observed Rainfall,	Rainfall, in Percentage of Mean Annual Quantity,

TABLE 5.—REPRESENTATIVE PRECIPITATION STATIONS AND CHARACTERISTICS, FROM TABLE 1 (APPENDIX I).

Station. No.	Station.	Latitude zone.	Relation to coast.	Relative elevation.	Mean month.	Record of Observed Rainfall, in Inches.					Mean month.	Rainfall, in Percentage of Mean Annual Quantity.				
						Maximum.		Annual.				Maximum.		Annual.		
						24 hours.	Month.	Minimum.	Mean.	Maximum.		24 hours.	Month.	Minimum.	Mean.	Maximum.
784	Verkhoyansk, Siberia	Above 60°	Interior	Low	0.9	0.8	3.4	2.0	5.0	9.1	18	16	68	40	100	182
788	Obdorsk, Siberia	"	"	"	2.0	....	5.8	4.5	10.1	14.4	20	....	57	44	100	142
791	Bogoslovsk, Siberia	"	"	"	2.9	2.6	7.7	5.4	16.4	28.4	17	16	47	33	100	173
418	Dawson, Canada	"	"	High	1.2	....	4.2	9.2	14.4	17.7	8	....	29	64	100	123
782	Novo-Mariinski, Siberia	"	Coastal	Low	1.5	....	4.3	4.7	7.4	10.3	21	....	58	64	100	139
804	Gjesvar, Norway	"	"	"	3.3	....	10.4	17.3	29.1	46.1	11	....	36	59	100	158
811	Vestmanno, Iceland	"	"	"	5.8	....	13.9	37.3	53.7	70.7	11	....	26	69	100	131
313	Yuma, Ariz	30° to 60°	Interior	Low	0.6	4.0	6.2	0.6	3.4	11.4	17	117	182	18	100	335
185	St. Paul, Minn	"	"	"	4.1	5.7	11.7	10.2	27.2	49.7	15	21	43	37	100	183
142	Chattanooga, Tenn	"	"	"	4.8	7.6	15.3	32.7	51.3	70.6	9	15	30	64	100	138
266	Reno, Nev	"	"	High	1.6	2.7	6.8	4.7	8.2	13.7	20	33	83	57	100	167
256	Tamarack, Calif	"	"	"	11.6	....	39.8	27.8	50.6	94.3	23	....	79	55	100	187
638	Santis, Switzerland	"	"	"	8.7	....	27.5	53.3	100.3	154.2	9	....	27	58	100	154
236	San Diego, Calif	"	Coastal	Low	1.9	3.2	9.3	4.3	9.8	27.6	19	33	95	44	100	283
223	Fort Ross, Calif	"	"	"	11.4	....	37.4	21.7	51.9	88.2	22	....	72	42	100	170
361	Valdivia, Chile	"	"	"	16.3	....	34.0	71.7	106.3	140.7	15	....	32	67	100	132
570	O'okiep South Africa	"	"	High	1.1	....	4.3	2.1	6.7	11.7	17	....	64	31	100	174
13	Mt. Washington, N. H	"	"	"	10.3	....	23.9	55.8	82.2	121.1	13	....	29	68	100	147
717	Ben Nevis, Scotland	"	"	"	19.1	7.3	48.3	107.8	160.8	240.1	12	4	30	67	100	150
611	Assuan, Egypt	10° to 30°	Interior	Low	0	0	0	0	0	0	....	....	....	....	....	....
601	Timbuktu, West Africa	"	"	"	2.8	....	....	....	7.7	....	36	....	....	....	100	....
373	Asuncion, Paraguay	"	"	"	6.2	....	18.6	22.6	51.8	78.5	12	....	36	44	100	151
549	Khartoum, Egypt	"	"	High	2.6	3.5	7.4	1.3	5.7	14.4	46	61	130	23	100	253
550	Deim Zubeir, Egypt	"	"	"	7.8	5.4	23.9	....	45.7	78.5	17	12	52	....	100	172
560	Belvedere South Africa	"	"	"	16.5	....	....	....	81.4	....	20	....	....	....	100	....
586	Swakopmund, South West Africa	"	Coastal	Low	0.1	....	0.8	0.1	0.6	1.2	17	....	133	17	100	200
108	Miami, Fla	"	"	"	9.1	9.0	27.9	33.1	60.0	89.1	15	15	46	55	100	148
511	Akyab, Burma	"	"	"	53.7	....	97.8	142.0	203.8	323.5	26	....	48	70	100	158
352	Arequipa, Peru	"	"	High	1.7	....	8.1	0.2	4.3	12.0	40	....	188	5	100	279
492	Baguio, Philippine Islands	"	"	"	40.3	46.0	133.0	125.7	182.0	355.8	22	25	73	69	100	196
515	Cherra Punji, India	"	"	"	107.4	40.0	366.1	282.6	456.9	905.+	23	9	80	62	100	198+
593	Gazi, Congo	0° to 10°	Interior	Low	8.5	....	....	....	72.7	....	12	....	....	....	100	....
594	Eala, Congo	"	"	"	8.8	....	....	....	70.5	....	12	....	....	....	100	....
552	Adis Ababa, Abyssinia	"	"	High	7.5	3.0	22.5	....	49.6	74.9	15	6	45	....	100	151
553	Entebbe, Uganda	"	"	"	8.5	....	15.8	47.2	58.0	75.7	17	....	27	81	100	131
551	Gore, Abyssinia	"	"	"	11.3	3.5	17.0	....	75.7	92.6	15	5	22	....	100	122
442	Malden Island, Pacific	"	Coastal	Low	4.7	....	25.7	3.9	28.6	93.6	17	....	90	14	100	327
554	Dar-es-Salaam, East Africa	"	"	"	11.8	....	23.8	19.4	42.3	56.9	28	....	56	46	100	134
602	Freetown, Sierra Leone	"	"	"	36.6	....	61.1	102.5	157.2	206.6	23	....	39	65	100	132
523	Nuwara Eliya, Ceylon	"	"	High	11.9	....	35.0	68.4	92.8	131.5	13	....	38	74	100	142
477	Kajoemas, East Indies	"	"	"	17.7	....	40.3	70.4	99.4	154.0	18	....	41	71	100	155

tabulations, when summarized data such as those in Table 1 (Appendix I) would serve more satisfactorily.

Arrangement of a long series of natural phenomena according to magnitude of the observed quantities, such as are used in precipitation, and run-off records, invariably results in a crowding near the mean values, and a thinning out toward the extremes, suggesting a mass distribution like that of a cone or of a pyramid, in which the base represents the mean and the apex, either the maximum or the minimum. It is naturally expected that the longer the period of record, the wider is the interval between the mean and either the maximum or the minimum. Because the 500-year or other long-period maximum must fall in the record of some decade, care should be taken to investigate such a possibility. When such happenings are properly taken into account, some interesting and useful relations become apparent regarding periodic maxima and frequency curves:

First.—The only direct relations to be observed consistently among station records are such as may be ascribed either to contiguity or to similarity of climatic conditions.

Second.—No direct relation is traceable between rainfall and sunspot periodicity (11 to 12 years), which closely corresponds to the time of revolution about the sun (11.86 years) of the planet, Jupiter, representing 71% of the total known mass of the solar planetary system. Seven of the longest known annual precipitation records are platted in Fig. 2, for convenient comparison. The occasional similarities, contrasts, and synchronisms therein displayed, seem to indicate endless variety rather than conformity to any generalization.

Third.—The mean monthly or the mean annual precipitation, together with the maxima for corresponding periods to be expected during average intervals of 10 years, 50 years, and 250 years, seems to form a simple arithmetical progression, while the related time intervals, namely, 2, 10, 50, and 250 years, form a geometrical progression. Fig. 3 illustrates for the yearly precipitation how nearly the straight-line relation holds for frequency curves when they are thus plotted on semi-logarithmic paper. Similar results are found for monthly periods.

Fourth.—The ten percentage columns of Table 1 (Appendix I) seem to provide generally ascending series of either geometrical or arithmetical progression, from left to right. The quantities in Column (1), headed "Mean Daily Intensity on Rainfall Days", are but the reciprocals of the average number of rainy days per year, expressed in percentage. Thus, for stations with 90 days of rainfall per annum, the mean daily intensity is 1.1% of the total mean annual precipitation. The quantities in Column (1) seem to be significant and important when considered in connection with the remaining nine columns. In general, this percentage is numerically equal to the maximum rainfall to be expected in a 3-min. period.

Since the beginning of these investigations and the assembling of the foregoing data, many opportunities have been presented for practical tests of

methods herein advocated. For example, preliminary estimates were made for the waterways required in connection with current projects, mainly from consideration of rainfall habits and soil texture, classification, and covering. Later they were compared with the final estimates, made after detailed study of the water-shed, run-off records, and all other available data, with satisfactory results.

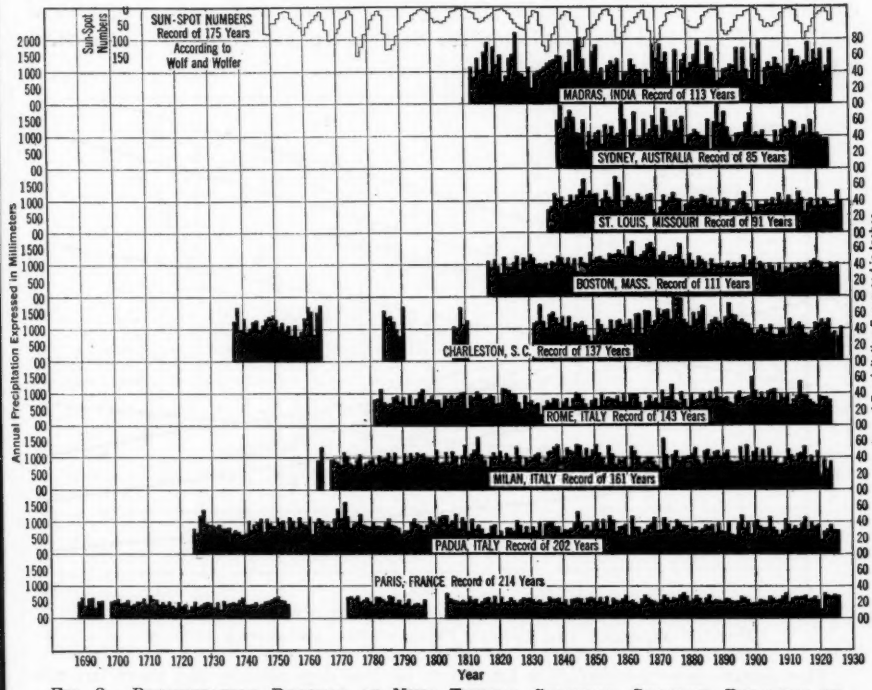
Sun-Spot Numbers
SUN-SPOT NUMBERS
Record of 175 Years
According to
Wolf and Wolfer
MADRAS, INDIA Record of 113 Years
SYDNEY, AUSTRALIA Record of 85 Years
ST. LOUIS, MISSOURI Record of 91 Years
BOSTON, MASS. Record of 111 Years
CHARLESTON, S.C. Record of 137 Years
ROME, ITALY Record of 143 Years
MILAN, ITALY Record of 161 Years
PADUA, ITALY Record of 202 Years
PARIS, FRANCE Record of 214 Years
Annual Precipitation Expressed in Millimeters
Annual Precipitation Expressed in Inches
Year

FIG. 2.—PRECIPITATION RECORDS AT NINE TYPICAL STATIONS, SHOWING RELATION TO SUNSPOT NUMBERS.

Application to a foreign problem is afforded by the Habra Dam disaster of November 26, 1927, in the Province of Oran, Northern Algeria. This dam had previously failed on March 10, 1872, in the year following its construction; and again on December 16, 1881, after reconstruction and enlargement of the spillway, the fatalities in this instance amounting to 400.*

The scarcity or utter absence of meteorological and hydrographic records are admittedly among the primary causes of the recurrent disasters. Through costly experience it has now been demonstrated that the 3 085 sq. miles of water-shed above the dam site are reasonably capable of yielding run-off at the rate of 20 sec-ft. per sq. mile instead of only one-fourth or one-half of that amount as assumed for the original construction and reconstruction, respectively.

* "The Failure of the Habra Dam, Algeria," *Le Génie Civil*, March 17, 1928, p. 256.

There is enough similarity observable in the rainfall patterns for Italian, French, and Spanish stations near the Mediterranean Coast, to permit expansion of the records for North Africa by use of such definite data as have long been available for the Ports of Tunis, Algiers, and Tangiers. The annual rainfall to be expected in the vicinity of Oran and Perregaux, in the Habra Basin, is known to range usually from 20 to 28 in.; the monthly maximum is to be expected in the winter season, or in the early spring, and may amount to as much as 12 in.; and the maximum for 24 hours might be safely assumed as 6 in.

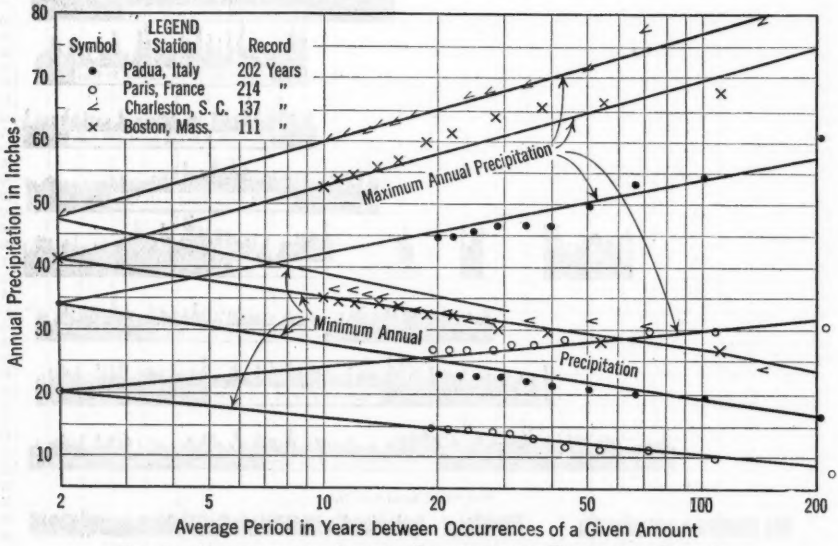

FIG. 3.—MONTHLY AND ANNUAL PRECIPITATION FREQUENCY CURVES.

The occurrence of rainfall is known to be less dependable and more intense in Algeria than on the northern shores of the Mediterranean, just as should be expected when the desert character of the hinterland is taken into account. Finally, the Tiber River in Italy and the rivers of Eastern Spain should have afforded an index as to the expected run-off from the Habra Basin capable of governing an adequate design for the spillway that has recently failed for the third successive time.

Illustrative of the recommended method of approach for such practical problems, the meteorological records of the most nearly adjacent and comparable stations were reviewed in their summarized form as used in Table 1 (Appendix I), and supplemented by such literature and data as were available for the locality, in order to form a fair conception of the run-off habits to be expected under extreme conditions, as well as the average prevailing in that locality. Other published data* afford a number of examples which may be

* "Flood Flow Characteristics," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1003, Table 2 (Appendix I).

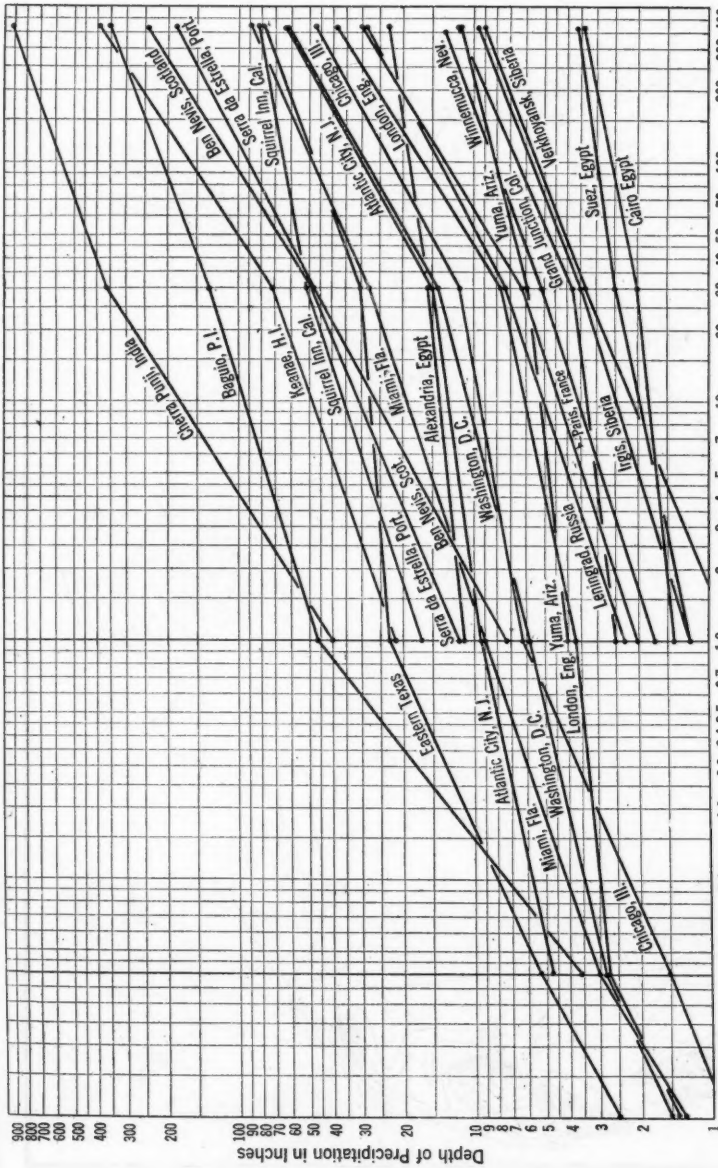

FIG. 4.—CHART OF MAXIMUM RAINFALL OBSERVED AT TWENTY-TWO TYPICAL STATIONS FOR VARIOUS TIME INTERVALS (BASED ON DATA IN TABLE 1 (APPENDIX I)).

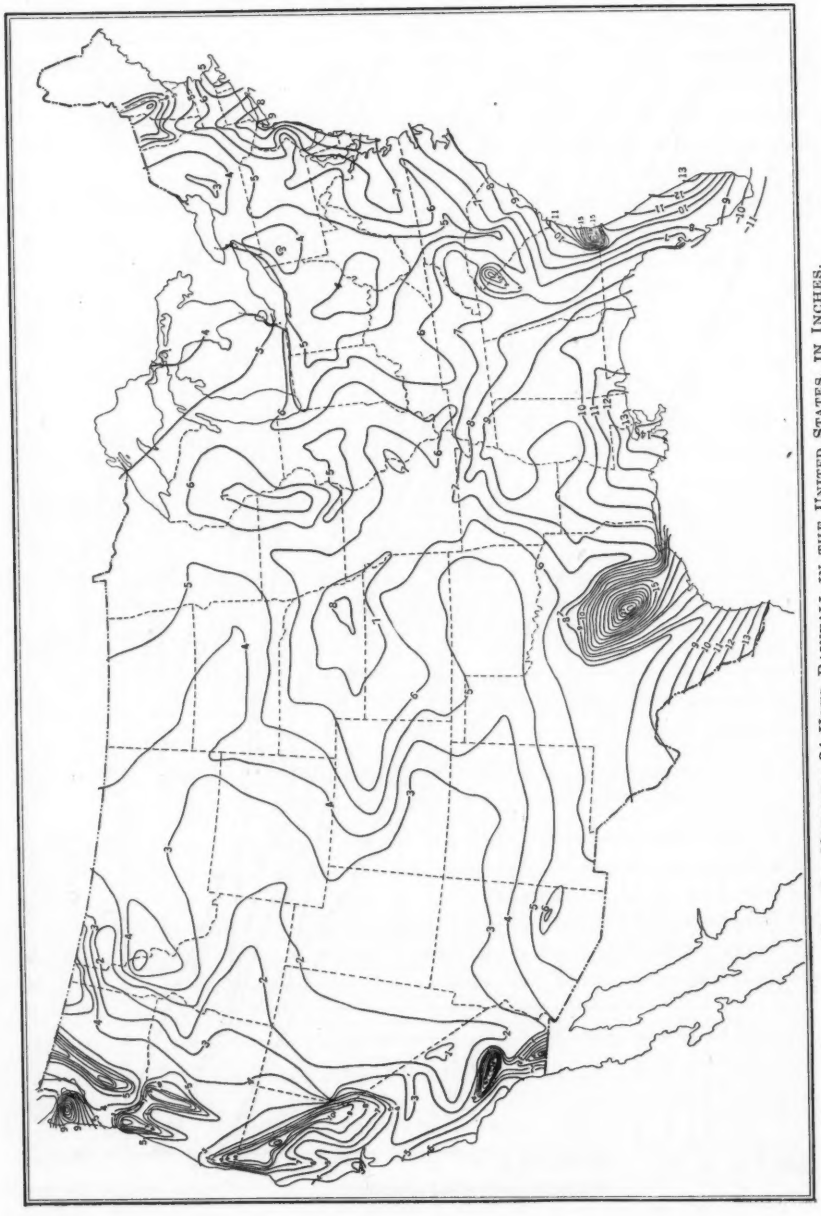

FIG. 5.—MAXIMUM 24-HOUR RAINFALL IN THE UNITED STATES, IN INCHES.

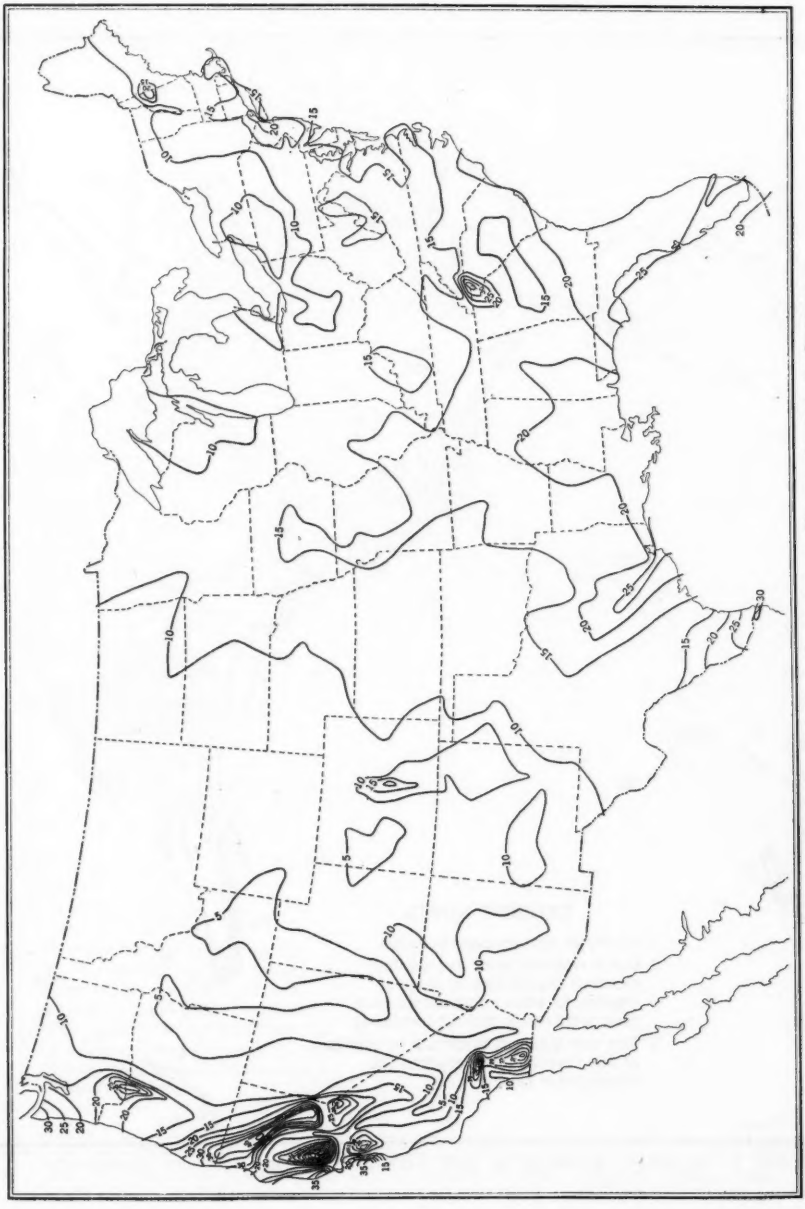

FIG. 6.—MAXIMUM MONTHLY RAINFALL IN UNITED STATES, IN INCHES.

FIG. 5.—MAXIMUM 24-HOUR RAINFALL IN THE UNITED STATES, IN INCHES.

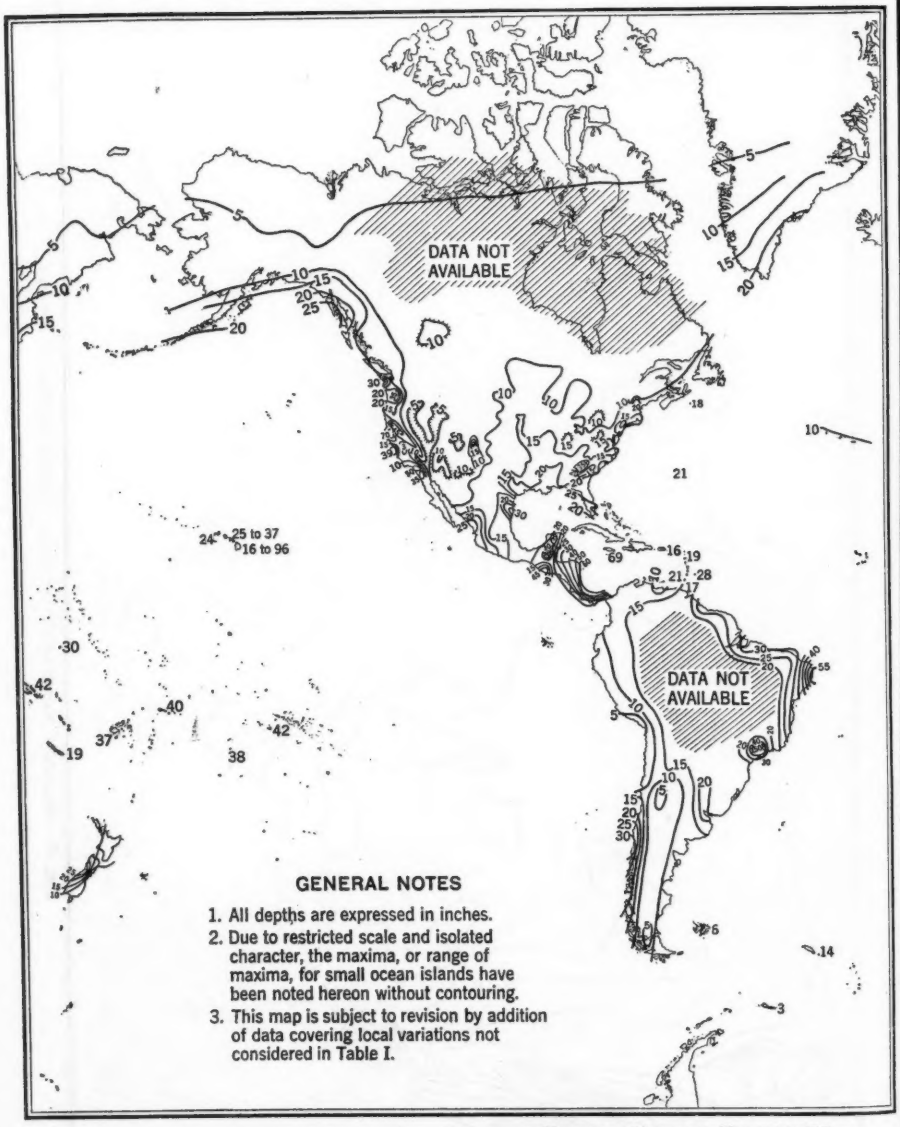
DATA NOT AVAILABLE
DATA NOT AVAILABLE
25 to 37
16 to 96
GENERAL NOTES
1. All depths are expressed in inches.
2. Due to restricted scale and isolated character, the maxima, or range of maxima, for small ocean islands have been noted hereon without contouring.
3. This map is subject to revision by addition of data covering local variations not considered in Table I.

FIG. 7.—MAXIMUM RAINFALL IN ANY CALENDAR MONTH—WESTERN HEMISPHERE.

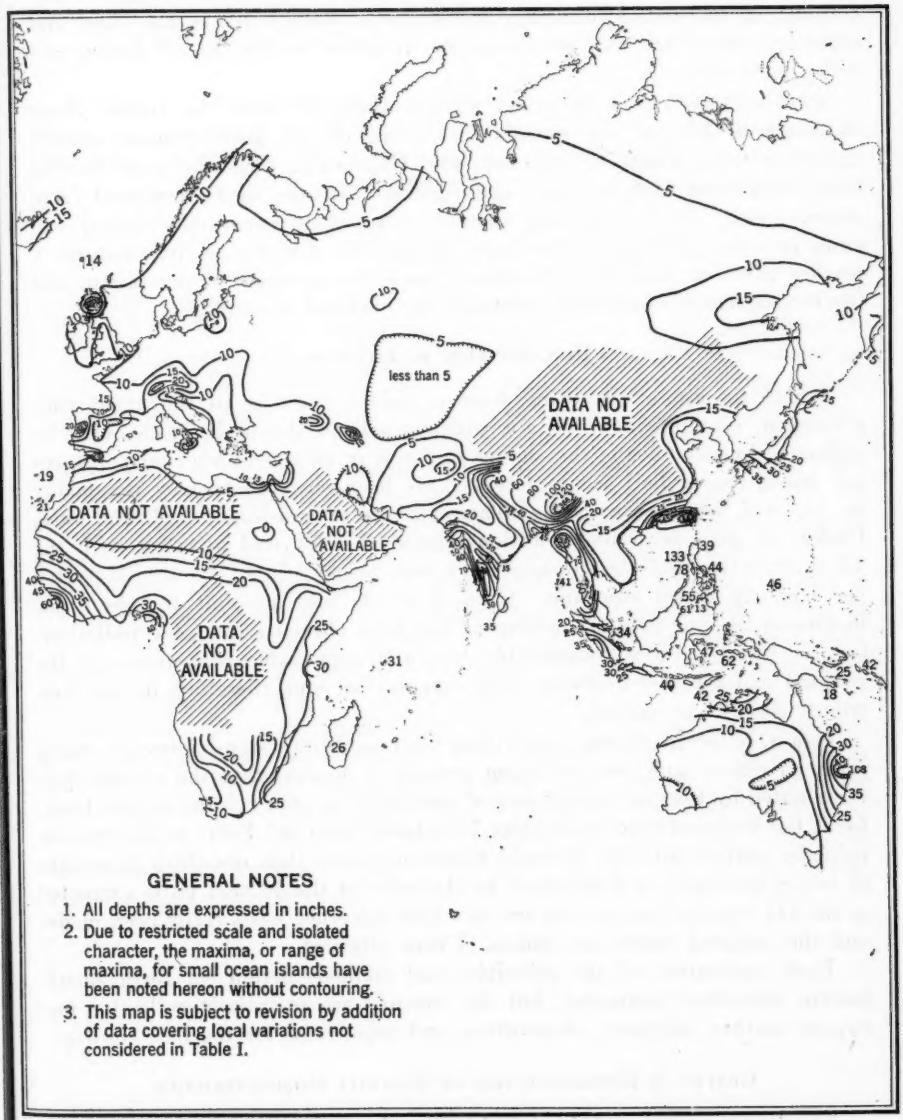
less than 5
DATA NOT AVAILABLE
DATA NOT AVAILABLE
DATA NOT AVAILABLE
DATA NOT AVAILABLE
GENERAL NOTES
1. All depths are expressed in inches.
2. Due to restricted scale and isolated character, the maxima, or range of maxima, for small ocean islands have been noted hereon without contouring.
3. This map is subject to revision by addition of data covering local variations not considered in Table I.

FIG. 7.—MAXIMUM RAINFALL IN ANY CALENDAR MONTH—EASTERN HEMISPHERE.

utilized for comparisons if due allowance be made for differences in the temperature, humidity, seasonal habits, and wind directions during stormy periods; in the soil, topography, and shape of basin; and in the slope and other influences that may determine the retention or the run-off during and following a storm.

Obviously, for an estimate of stream discharge from the Habra River of Northern Algeria, the records from some of the Mediterranean coastal rivers, or from Virginia, Arkansas, and California, would be more helpful than those from high latitudes and rigorous climates, or far removed from desert areas. Neither ice nor snow accumulations nor deeply frozen soil enter into the problem for Northern Africa; the periodic aridity produces a marked effect on both the soil structure and the character of vegetation, and the run-off habits of drainage channels are modified accordingly.

Run-Off Due to Thawing

Where the run-off resulting from a sudden thaw is an important consideration, the limiting rates for which provision should be made, may be estimated by reference to the seasons of the year when both high temperatures and snow, ice, or hail accumulations have been observed or may reasonably be expected, together with the relative effects of sunshine, wind, and rain. Under the most favorable conditions conceivable for mid-latitudes, with the air clear and dry and the sun near the zenith, nearly $\frac{1}{2}$ in. of ice may be melted per hour by direct sunshine. Chinook winds may account for the same maximum 24-hour rate of melting as has been observed for solar radiation; but the hourly rate is considerably less, and appreciable percentages of the released moisture are disposed of by evaporation, and, therefore, do not contribute directly to run-off.

Again, if the maximum conceivable flood conditions seem to involve warm rains on either snow, ice, or frozen ground, it appears that the volume thus contributed by thawing cannot exceed one-third the rain volume at that time; for if the temperature of rain water be reduced from 80° Fahr. to the freezing point by contact with ice, it would release not more than one-third its weight of frozen moisture, as determined by the ratio of the 48 heat units extracted to the 144 required for melting ice, to which reference has already been made; and the assumed limits are seldom if ever attained.

Each application of the principles and methods herein set forth should receive individual treatment; but the controls are essentially collective and require mature judgment, observation, and experience for the best results.

Graphical Representation of Rainfall Characteristics

The maximum rates of rainfall that have been observed as typical stations representing almost the entire range of habits for the stations listed in Table 1 (Appendix I), are illustrated in Fig. 4. The platting on multi-phase logarithmic diagrams brings out not only the similarities, but also the diver-

gencies of precipitation patterns and trends. Figs. 5 and 6 portray the maximum observed depths of precipitation in 24 hours and in one calendar month, respectively, for the United States, based on the data of Table 1 (Appendix I). Fig. 7 is a graphical representation of the maximum monthly rainfall over the entire earth so far as is disclosed by the data contained in Table 1 (Appendix I).

It is readily recognized that the inclusion of additional station records and the use of a larger scale for mapping might introduce important revisions. The curves drawn on the Mercator projection, however, may serve as generalizations and may assist in visualizing some of the rainfall characteristics of the world.

Conclusions

1.—The precipitation to be reasonably expected at a given place, together with the run-off resulting therefrom, may be regarded as but the expression of certain physical forces and habits, explainable in the main on the basis of known scientific laws and principles applied to the source of supply and to the paths traversed by winds and drainage channels.

2.—The factors responsible for both rainfall and run-off are fairly well defined; most of the earth's surface has been explored; the courses of ocean currents and the prevalent paths of storms have been charted; and the potentialities of recurrent storm movements have been recorded in terms of barometric pressures, precipitation, wind velocities, and flood volumes.

3.—Wherever meteorological and hydrographic records are available, approximations may be made with varying degrees of dependability as to both the ordinary habits and the extremes of behavior against which provision should be made. By utilizing the long-period records as controls, the brief or fragmentary data may be interpreted more effectively.

4.—Precipitation and run-off habits which govern the design of waterways display characteristics definite enough to be classified, and they are known or may be derived from well co-ordinated records within nearly the same range of accuracy as is ordinarily attained for strength of structural materials.

5.—Many of the catastrophes resulting from storms and floods should be recognized as preventable accidents, and public works should be inaugurated to reduce such hazards.

APPEN-

TABLE 1.—Observed Rain-
(Depths Express-

Station No.	Station.	Elevation above sea, in feet.	Years of Record. For day or less.	Years of Record. For month and longer periods.	Years of Record. Last year included.	Maximum Observed Precipitation. Number of Minutes. 5.	15.	60.	Day of 24 hours.	Month.	Annual Precipitation. Minimum.	Mean.	Maximum.	Observed Precipitation, in Percentages of Mean Annual Rainfall. Mean daily fluctuation (per cent of rainfall).	Maximum During Short Periods. Number of Minutes. 5.	15.	60.	Mean for month of maximum record.	Maximum. In 24 hours.	Maximum. In one calendar month.	Annual. Minimum.	Annual. Mean.	Annual. Maximum.	Main references.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
1	Eastport, Me.	53	....	58	1927	....	....	....	....	12.2	22.8	40.4	64.5	0.7	....	....	....	7.8	....	32.6	67	100	160	2; 3
2	Bar Harbor, Me.	20	....	44	1920	....	....	....	....	14.4	32.1	48.3	63.9	0.8	....	....	....	10.3	....	29.8	66	100	142	5
3	Orono, Me.	129	....	56	1925	....	....	....	....	10.5	26.0	41.7	53.8	0.9	....	....	....	7.7	....	25.2	62	100	129	5
4	Upper Oxford Dam, Me.	1 470	....	40	1925	....	....	....	....	10.8	22.4	32.8	41.7	....	....	....	....	10.1	....	26.4	68	100	127	5
5	Lewiston, Me.	185	....	52	1927	....	....	....	....	12.3	32.7	44.2	58.6	....	....	....	....	9.8	....	27.8	74	100	133	2; 3
6	Gardiner, Me.	163	....	99	1927	....	....	....	....	13.1	30.2	43.0	54.6	....	....	....	....	9.5	....	30.5	70	100	127	2; 3
7	Portland, Me.	98	59	67	1927	0.42	1.09	1.58	5.3	10.2	22.0	41.7	59.7	0.8	1.0	2.6	3.8	7.9	12.7	29.2	77	100	142	5
8	Cornish, Me.	778	....	67	1926	....	....	....	....	12.3	33.4	46.1	63.8	....	....	....	....	8.1	....	26.6	72	100	138	5
9	Lakeport, N. H.	500	....	69	1925	....	....	....	....	12.6	31.5	42.2	55.6	....	....	....	....	9.1	....	29.9	75	100	132	5
10	Concord, N. H.	500	....	70	1927	0.55	1.65	2.48	4.1	11.6	29.2	38.4	54.3	0.5	1.4	4.0	6.5	8.7	10.7	30.2	68	100	141	2; 3
11	Hanover, N. H.	603	....	81	1927	....	....	....	....	11.8	22.7	35.9	55.8	0.8	....	....	....	9.6	....	27.5	63	100	155	2; 3
12	Plymouth, N. H.	500	....	40	1927	....	....	....	....	11.5	30.8	39.3	53.5	....	....	....	....	8.9	....	29.3	78	100	136	2; 3
13	Mt. Washington, N. H.	6 293	....	21	1890	....	....	....	....	23.9	55.8	92.2	121.1	....	....	....	....	12.6	....	29.0	68	100	147	2
14	Bethlehem, N. H.	1 470	....	33	1925	....	....	....	....	9.8	28.0	36.7	46.1	....	....	....	....	8.7	....	29.6	76	100	125	5
15	Burlington, Vt.	404	22	93	1927	0.44	0.78	1.52	4.5	9.9	24.3	32.6	49.4	0.7	1.3	2.4	4.7	10.6	13.7	30.4	74	100	152	5; 3
16	Northfield, Vt.	876	....	40	1927	0.45	1.64	1.91	7.6	11.4	27.4	33.2	45.9	0.6	2.6	5.0	5.8	8.5	22.9	34.4	83	100	138	2; 3
17	Chelsea, Vt.	840	30	42	1927	....	....	....	4.5	10.9	27.1	35.8	48.1	....	....	....	....	8.7	12.5	30.5	76	100	134	2; 3
18	Woodstock, Vt.	700	....	60	1927	....	....	....	....	12.9	29.7	38.2	51.3	....	....	....	....	9.0	....	33.8	78	100	134	2; 3
19	Williamstown, Mass.	711	....	50	1926	....	....	....	....	14.8	30.5	38.1	54.5	0.9	....	....	....	11.2	....	38.8	80	100	143	5; 3
20	Springfield, Mass.	200	....	79	1926	....	....	....	....	15.0	30.9	39.7	58.0	....	....	....	....	9.9	....	34.4	71	100	132	5; 3
21	Amherst, Mass.	272	....	91	1926	....	....	....	....	14.5	31.0	43.9	58.0	0.8	....	....	....	9.9	....	33.0	71	100	132	5; 3
22	Lowell, Mass.	80	....	101	1926	....	....	....	....	12.3	27.8	41.3	59.5	....	....	....	....	9.9	....	29.8	67	100	144	5; 3
23	Boston, Mass.	126	57	110	1927	0.55	1.12	1.64	6.0	12.4	27.2	43.6	67.7	0.8	1.3	2.6	3.8	8.2	13.8	29.6	63	100	158	1
24	Cambridge, Mass.	40	....	105	1927	....	....	....	....	....	26.5	42.3	62.5	....	....	....	....	....	....	....	68	100	148	5
25	Cambridge, Mass (Harvard Obs).	75	....	81	1921	....	....	....	....	15.0	30.1	43.5	59.3	....	....	....	....	9.5	....	34.5	69	100	137	5
26	Cambridge, Mass (Fresh Pond).	40	....	40	1926	....	....	....	....	14.0	30.0	43.5	59.3	....	....	....	....	9.5	....	34.5	69	100	137	5
27	Waltham, Mass.	30	....	101	1925	....	....	....	....	14.0	28.4	42.3	62.1	....	....	....	....	9.7	....	33.2	67	100	147	5
28	Lake Cochituate, Mass.	140	....	74	1925	....	....	....	....	13.3	31.2	45.6	69.2	....	....	....	....	8.5	....	29.1	68	100	152	2; 5
29	New Bedford, Mass.	100	....	113	1926	....	....	....	....	13.2	28.0	40.2	61.4	....	....	....	....	8.5	....	29.6	76	100	153	5; 3; 15
30	Nantucket, Mass.	15	33	52	1926	0.44	0.83	2.26	5.7	11.0	27.1	39.3	58.0	0.7	1.1	2.1	5.8	9.2	14.5	28.0	69	100	146	5; 3; 15
31	Block Island, R. I.	26	....	46	1926	0.21	0.52	1.53	....	9.8	27.2	41.2	63.1	0.8	0.5	1.3	3.7	8.4	....	23.8	66	100	163	5; 3; 15
32	Providence, R. I.	162	22	95	1926	0.45	1.11	1.98	4.8	11.3	30.5	45.1	63.5	0.8	1.0	2.5	4.4	8.6	10.6	25.0	68	100	141	9; 1; 5
33	Norwich, Conn.	150	....	55	1925	....	....	....	....	11.3	29.8	44.1	62.8	....	....	....	....	9.8	....	29.8	67	100	141	5
34	New London, Conn.	47	....	56	1926	....	....	....	....	16.4	30.0	44.1	60.6	....	....	....	....	9.6	....	37.2	68	100	137	5; 2; 3
35	Middletown, Conn.	125	....	67	1925	....	....	....	....	15.5	34.0	46.8	66.7	....	....	....	....	8.5	....	33.1	73	100	142	5
36	Hartford, Conn.	159	....	55	1926	0.49	1.30	2.64	....	15.3	33.5	44.7	61.0	0.8	1.1	2.9	5.9	9.7	....	34.2	75	100	136	1
37	Canton, Conn.	900	....	64	1926	....	....	....	....	16.4	34.0	48.8	72.5	0.9	....	....	....	8.6	....	34.0	71	100	154	5
38	New Haven, Conn.	127	....	76	1926	0.40	1.06	2.33	....	17.1	33.9	45.6	60.3	0.8	0.9	2.3	5.1	9.4	....	37.6	74	100	132	5; 2; 3
39	New York, N. Y.	314	26	56	1927	0.75	1.63	2.48	....	14.5	33.2	43.1	58.7	0.8	1.7	3.8	5.8	7.9	....	33.7	77	100	136	1
39-A	New York, N. Y.	314	56	101	1927	....	....	....	9.4	14.3	33.2	42.1	58.7	0.8	....	....	....	7.9	26.3	34.4	77	100	136	2; 1
40	Yonkers, N. Y.	100	....	48	1925	....	....	....	....	22.7	25.2	48.4	85.2	....	....	....	....	10.4	....	47.0	52	100	176	5
41	West Point, N. Y.	167	....	76	1925	....	....	....	....	13.5	31.5	45.3	63.6	1.0	....	....	....	8.1	....	29.8	70	100	141	2; 5
42	Boyd's Corners, N. Y.	600	....	58	1925	....	....	....	....	14.4	35.3	49.0	68.4	....	....	....	....	9.6	....	29.4	72	100	129	2; 5
43	Albany, N. Y.	97	45	101	1926	0.57	1.17	1.83	4.7	13.5	28.1	37.4	47.3	0.7	1.5	3.2	4.9	8.2	12.5	36.0	73	100	153	2; 5; 1
44	Plattsburgh, N. Y.	163	....	73	1926	....	....	....	....	12.3	29.6	38.8	47.3	1.0	....	....	....	11.6	....	30.2	69	100	129	5
45	Canton, N. Y.	448	15	38	1920	....	....	....	....	3.5	8.9	28.3	36.0	46.2	....	....	....	9.5	9.7	24.7	78	100	126	2
46	Oswego, N. Y.	335	50	74	1920	....	....	....	....	3.8	9.8	23.4	36.5	56.1	0.6	....	....	8.9	10.4	26.8	64	100	154	2
47	Syracuse, N. Y.	397	19	24	1920	0.56	1.16	2.65	....	4.8	9.4	29.2	41.3	57.6	1.6	3.3	7.6	8.5	12.8	32.4	61	100	154	2; 15
48	Rochester, N. Y.	500	50	92	1920	....	....	....	....	4.2	8.7	17.0	33.2	45.9	0.6	....	....	8.8	12.7	26.2	51	100	154	2
49	Fort Niagara, N. Y.	250	....	71	1920	....	....	....	....	3.5	7.9	18.4	30.2	44.6	0.7	....	....	10.2	....	24.1	63	100	127	2
50	Buffalo, N. Y.	824	....	71	1927	0.79	....	....	....	6.2	10.6	25.3	39.1	60.2	0.6	2.2	....	8.3	11.6	29.3	70	100	167	2; 1; 8
51	Jamestown, N. Y.	1 365	....	28	1918	....	....	....	....	11.2	34.4	43.3	52.7	0.6	....	....	....	13.0	....	25.9	79	100	122	2
52	Bolivar, N. Y.	1 800	....	32	1920	....	....	....	....	11.2	31.5	40.1	48.6	0.7	....	....	....	9.7	....	26.4	77	100	122	2
53	Ithaca, N. Y.	928	42	64	1915	....	....	....	4.2	8.4	29.2	32.9	46.4	0.6	....	....	....	9.7	12.5	25.6	62	100	141	2
54	Cooperstown, N. Y.	1 200	....	37	1920	....	....	....	....	10.2	29.9	40.9	58.4	0.7	....	....	....	10.6	....	27.0	68	100	154	2
55	Binghamton, N. Y.	871	....	38	1927	....	1.00	2.03	4.8	8.1	22.4	34.0	42.6	0.7	....	2.9	5.9	10.3	13.2	23.7	66	100	125	1
56	Scranton, Pa.	805	....	33	1927	....	....	....	....	5.1	9.2	31.3	37.3	45.3	0.7	....	....	8.3	13.7	24.8	84	100	122	1; 2
57	Philadelphia, Pa.	156	46	56	1927	....	....	2.35	5.9	12.1	29.3	41.2	55.3	0.7	....	....	....	11.2	14.5	29.4	71	100	134	1; 2
58	Philadelphia, Pa.	156	46	101	1926	....	....	2.35	5.9	12.1	29.3	41.2	55.3	0.7	....	....	5.5	10.5	14.5	29.4	71	100	134	2
59	West Chester, Pa.	455	59	72	1920	....	....	....	6.6	14.6	33.9	49.1	73.0	0.7	....	....	....	9.8	13.8	29.7	67	100	149	2
60	Reading, Pa.	322	....	53	1920	....	....	....	4.2	9.6	21.4	40.9	58.4	0.7	....	....	....	9.8	13.2	29.6	74	100	146	2; 1
61	Lebanon, Pa.	469	26	82	1920	....	....	....	5.2	12.9	28.9	44.4	64.1	0.8	....	....	....	10.4	11.7	29.0	65	100	145	2
62	Harrisburg, Pa.	361	32	55	1920	....	....	....	6.2	13.6	26.0	39.4	57.1	0.8	....	....	....	9.7	15.8	34.7	64	100	146	2
63	Altoona, Pa.	1 631	....	22	1920	....	....	....	4.2	9.6	21.4	40.9	58.4	0.7	....	....	....	11.4	13.2	29.6	74	100	146	2
64	Johnstown, Pa.	1 184	28	40	1920	....	....	....	4.5	12.2	37.5	47.8	58.7	0.6	....	....	....	9.6	9.4	25.5	78	100	123	2
65	Freeport, Pa.	750	....	44	1920	....	....	....	....	8.9	30.8	42.0	57.9	....	....	....	....	7.9	....	21.2	73	100	138	2
66	Franklin, Pa.	955	25	55	1920	....	....	....	2.8	11.6	34.0	42.5	59.7	0.6	....	....	....	10.5	6.6	27.3	80	100	140	2
67	Erie, Pa.	714	46	46	1920	....	....	....	3.2	7.1	24.0	34.0	47.2	0.6	....	....	....	8.1	10.6	26.1	71	100	140	2
68	Pittsburgh, Pa.	842	57	87	1927	0.71	1.20	1.65	4.1	9.5	25.7	36.2	50.6	0.6	2.0	3.3	4.6	11.6	11.3	26.3	74	100	140	2
69	Pittsburgh, Pa.	842	57	101	1926	0.71	1.20	1.65	4.1	9.5	25.7	36.2	50.6	0.6	2.0	3.3	4.6	11.6	11.3	26.3	74	100	140	2; 3
70	Somerset, Pa.	2 280	....	42	1920	....	....	....	4.4	14.4	32.8	51.2	66.3	0.7	....	....	....	9.8	8.6	28.0	63	100	130	2
71	Morgantown, W. Va.	1 250	....	51	1923	....	....	....	....	11.2	28.1	43.3	69.5	0.9	....	....	....	11.5	....	25.8	65	100	161	2; 3
72	Parkersburg, W. Va.	638	25	72	1920	0.73	1.48	2.12	3.6	10.1	24.7	37.6	58.4	0.8	1.9	3.7	5.4	9.2	9.6	31.9	66	100	144	2; 3
73	Weston, W. Va.	1 026	....	37	1924	....	....	....	....	15.1	31.2	47.3	68.2	0.8	....	....	....	10.6	....	31.9	66	100	144	2; 3
74	Elkins, W. Va.	1 930	....	23	1927	0.56	1.02	2.37	4.4	12.1	27.4	43.1	62.1	1.2	2.2	5.1	7.2	11.2	9.2	30.2	73	100	152	2; 3
75	Washington, D. C.	112	57	85	1927	0.77	1.51	2.79	5.8	14.4	18.8	40.5	61.3	0.8	1.9	3.7	6.9	10.0	14.4	35.5	46	100	152	4; 3; 1
76	Washington, D. C.	112	53	53	1920	0.77	1.51	2.79	5.8	14.4	30.8	42.2	61.1	0.8	1.8	3.6	6.6	9.8	13.8	34.2	73	100	145	1
77	Baltimore, Md.	115	57	111	1927	....	....	....	....	6.1	12.3	32.3	48.3	0.8	....	....	....	9.9	15.3	39.1	75	100	147	2; 1
78	Baltimore, Md.	115	57	77	1927	....	....	....	....	6.1	12.3	32.3	48.3	0.8	....	....	....	10.0	14.4	39.1	75	100	147	2
79	Trenton, N. J.	60	28	43	1913	....	....	1.70	5.4	15.2	34.3	48.8	67.2	1.0	....	....	3.5	10.6	11.1	31.1	70	100	138	2
80	Newark, N. J.	159	....	18	1920	....	....	....	....	22.5	34.5	47.8	69.1	0.8	....	....	....	10.6	....	47.0	72	100	145	2
81	Atlantic City, N. J.	10	....	48	1927	....	....	4.73	9.2	14.9	29.1	46.0	61.1	0.8	....	....	11.6	10.7	22.7	36	100	156	2
82	Fortress Monroe, Va.	8	....	55	1890	....	....	....	....	18.0	19.3	42.6	74.1	1.0	....	....	....	10.9	....	42.2	45	100	174	2
83	Norfolk, Va.	91	57	57	1927	0.66	1.37	2.72	6.3	11.9	30.4	45.6	70.7	0.8	1.4	3.0	9.0	7.4	13.8	36.1	67	100	155	1
84	Norfolk, Va.	144	22	87	1927	0.70	1.52	2.97	7.1	13.6	31.6	47.1	71.8	0.8	1.7	3.7	7.1	8.1	17.6	33.6	67	100	155	1
85	Charlottesville, Va.	854	....	45	1920	....	....	....	....	14.4	30.1	44.3	65.3	0.9	....	....	....	10.4	....	32.1	67	100	146	2
86	Lexington, Va.	1 060	....	58	1920	....	....	....	....	12.4	30.2	42.0	60.3	0.9	....	....	....	9.8	....	30.2	64	100	144	2
87	Lynchburg, Va.	681	17	50	1920	0.56	1.51	3.49	7.2	12.4	30.5	41.7	60.6	0.8	1.5	3.6	8.4	9.8	17.2	29.7	73	100	145	2
88	Danville, Va.	413	....	39	1920	....	....	....	4.2	9.2	24.9	38.1	53.9	0.9	....	....	....	10.8	....	30.4	72	100	142	2
89	Wytheville, Va.	2 293	18	48	1920	....	....	....	....	4.2	13.1	30.9	40.4	62.5	0.8	....	....	10.6	10.4	32.5	76	100	155	2
90	Asheville, N. C.	2 255	18	19	1920	0.83	1.71	2.70	7.9	12.1	30.1	39.9	51.1	0.8	2.1	4.3	6.8	11.8	19.9	29.3	75	100	128	2
91	Murphy, N. C.	1 914	....	49	1920	....	....	....	....	15.4	45.6	59.2	84.8	0.9	....	....	....	8.0	....	26.0	77	100	143	2
92	Rock Hissar, N. C.	1 100	....	29	1920	....	....	....	....	26.4	62.9	84.1	[113.3] (110.6)	0.8	....	....	....	11.2	....	43.1	74	100	135	2
	Next to Maxima.																							
93	Charlotte, N. C.	778	18	44	1922	0.59	1.12	1.98	5.0	16.5	34.6	46.5	68.4	0.8	1.3	2.4	4.2	11.3	10.6	35.2	74	100	146	2
94	Raleigh, N. C.	390	41	41	1927	0.75	1.74	3.01	5.7	12.6	32.1	46.4	63.4	0.8	1.6	3.7	6.5	11.8	12.3	29.3	69	100	136	1
95	Hatteras, N. C.	11	....	25	1920	....	....	....	....	14.2	34.0	54.6	80.0	0.9	....	....	....	10.2	....	32.2	74	100	132	2
96	Wilmington, N. C.	52	50	50	1920	0.66	1.06	2.25	8.0	21.1	27.7	48.8	83.6	0.8	1.4	2.2	4.6	13.4	16.4	43.2	67	100	171	2
96-A	Effingham, S. C.	108	....	30	1920	....	....	....	....	13.2	24.9	34.3	48.9	0.9	....	....	....	11.8	27.0	50.9	70	100	144	2
97	Charleston, S. C.	46	23	126	1927	0.56	1.25	2.72	9.1	18.2	34.3	47.5	72.5	1.2	2.6	5.7	7.3	11.8	20.0	36.2	71	100	156	2; 1
98	Columbia, S. C.	351	20	48	1920	0.74	1.26	2.28	4.9	10.1	30.9	44.0	63.2	0.9	1.7	2.9	5.3	7.6	11.2	23.0	70	100	144	2
99	Camden, S. C.	222	....	32	1920	....	....	....	....	8.8	20.1	34.7	52.7	0.8	....	....	....	10.5	....	30.8	66	100	146	2
100	Greenville, S. C.	1 039	11	35	1920	0.52	1.23	2.43	8.2	19.5	42.7	52.0	77.8	0.9	1.0	2.4	6.8	9.7	15.8	37.5	82	100	149	2
100-A	Anderson, S. C.	764	26	....	1920	....	....	....	11.6	18.4	38.4	47.4	64.3	....	....	....	....	11.3	24.5	38.8	81	100	136	2
101	Augusta, Ga.	180	18	46	1920	1.24	2.11	3.08	5.6	14.0	33.8	46.5	60.7	0.9	2.7	4.5	6.6	10.6	15.2	30.1	77	100	139	2
102	Atlanta, Ga.	1 218	45	45	1920	....	....	....	7.7	13.5	33.1	46.2	60.3	0.9	....	....	....	10.3	15.2	31.2	71	100	138	2
103	Macon, Ga.	370	28	53	1926	....	....	....	6.8	13.4	31.7	44.9	61.9	0.9	....	....	....	10.8	15.2	29.8	71	100	136	2; 1
104	Savannah, Ga.	50	24	57	1927	0.72	1.21	1.56	3.8	12.8	29.2	45.8	73.9	0.9	1.5	2.5	7.4	11.2	20.8	37.2	80	100	152	2
104-A	St. George, Ga.	76	....	15	1920	....	....	....	18.0	24.2	39.0	49.6	63.2	....	....	....	....	17.7	26.8	48.8	62	100	128	2
105	Jacksonville, Fla.	222	33	56	1924	0.78	1.60	3.13	9.9	21.8	32.6	50.7	82.0	0.8	1.6	3.3	6.8	14.6	19.6	43.1	64	100	162	2
106	St. Augustine, Fla.	10	....	62	1923	....	....	....	....	23.9	31.6	47.1	71.8	1.0	....	....	....	10.6	....	50.0	66	100	162	2
107	Jupiter, Fla.	21	24	25	1929	0.65	2.12	3.83	13.2	21.4	39.8	59.8	87.1	0.7	1.7	3.6	6.4	17.2	22.3	56.0	67	100	146	2
108	Miami, Fla.	13	16	25	1929	0.58	1.34	3.06	6.0	27.9	34.1	60.0	89.2	0.8	1.0	2.2	5.3	15.2	15.0	46.6	71	100	146	2
109	Key West, Fla.	16	50	77	1920	0.65	1.40	3.35	11.9	19.8	20.5	37.3	69.7	0.9	1.7	3.9	9.9	13.7	31.9	53.0	55	100	187	2
	Next to Maxima.																							
110	Tampa, Fla.	103	31	48	1927	1.00	1.90	3.30	6.6	24.5	32.2	50.2	89.9	0.8	2.0	3.8	6.6	17.8	12.1	48.8	64	100	179	2; 1
111	Apalachicola, Fla.	24	24	24	1925	0.90	1.93	2.72	10.1	21.7	30.7	47.4	85.0	0.9	1.4	2.7	4.7	14.8	17.4	46.2	63	100	135	1
112	Pensacola, Fla.	151	33	33	1927	0.78	2.27	4.27	10.7	16.5	35.6	57.6	107.9	0.9	1.4	3.9	7.4	13.2	15.6	57.4	67	100	150	2
113	Mobile, Ala.	84	....	57	1927	....	....	....	....	26.7	39.5	61.8	92.3	0.8	....	....	....	8.9	....	43.2	64	100	149	1
114	Montgomery, Ala.	240	56	58	1927	....	....	....	10.0	17.8	37.0	51.7	77.9	0.9	....	....	....	10.1	19.4	34.5	72	100	151	1
115	Birmingham, Ala.	701	....	49	1927	....	....	....	8.8	20.1	34.3	52.7	76.2	0.8	....	....	....	10.5	16.8	38.5	66	100	146	2
116	Florence, Ala.	574	....	37	1920	....	....	....	....	16.1	40.7	51.2	68.4	1.1	....	....	....	11.2	....	31.4	79	100	134	2
117	Meridian, Miss.	375	32	34	1920	0.62	1.53	3.66	9.5	20.1	34.9	52.7	73.4	0.9	1.2	2.9	7.0	8.8	18.0	28.2	68	100	140	1
118	Vicksburg, Miss.	247	41	69	1920	0.83	1.61	3.11	8.6	22.3	38.0	52.0	84.2	0.9	1.6	3.7	6.0	10.3	15.4	42.7	72	100	162	2

DIX I.

FALL FOR VARIOUS PERIODS.
ed in Inches.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
1	3.75	...	9.0	3.24	...	9.4	3.88	...	9.4	2.96	...	6.8	3.16	...	13.2	3.11	...	7.4	3.15	...	9.1	3.23	...	9.4	2.91	...	6.4	3.91	...	9.2	3.55	...	8.6			
2	4.90	...	11.1	4.50	...	9.1	4.96	...	14.4	2.60	...	11.0	3.33	...	10.8	2.50	...	10.2	3.41	...	7.9	3.40	...	9.2	3.70	...	9.8	4.39	...	9.5	4.61	...	9.2	4.44	...	9.8
3	3.86	...	8.4	3.49	...	8.4	3.79	...	8.9	2.96	...	7.0	3.20	...	10.6	2.86	...	6.7	3.40	...	7.3	3.59	...	7.4	2.62	...	8.1	3.76	...	9.4	3.63	...	8.7	3.46	...	9.0
4	2.48	...	5.4	2.07	...	5.6	2.58	...	6.4	2.19	...	5.0	2.67	...	6.5	3.32	...	6.8	2.83	...	7.5	3.30	...	8.0	3.14	...	9.1	2.45	...	6.6	2.67	...	6.5	2.18	...	5.9
5	3.88	...	8.1	3.78	...	8.7	4.34	...	12.3	3.45	...	8.3	3.38	...	8.3	2.54	...	11.3	3.07	...	9.5	3.17	...	8.3	3.58	...	10.0	3.62	...	8.1	3.89	...	10.6	3.93	...	10.3			
6	3.62	...	7.9	3.44	...	9.3	3.91	...	10.1	3.37	...	7.9	3.05	...	11.8	3.24	...	8.9	3.37	...	9.2	3.47	...	8.5	3.26	...	8.2	4.08	...	12.1	3.97	...	10.6	3.81	...	9.4			
7	3.83	2.5	6.5	3.80	3.1	9.2	3.77	3.5	8.8	3.32	3.8	7.5	3.27	4.9	7.6	3.39	5.0	10.9	3.20	4.0	10.6	3.32	3.5	8.1	3.10	3.8	9.6	3.25	3.9	7.5	3.58	3.8	7.9	3.75	3.7	7.1
8	2.52	...	7.0	2.35	...	6.3	2.64	...	11.0	2.64	...	11.1	2.58	...	9.4	2.76	...	9.6	4.12	...	14.4	4.23	...	11.1	3.63	...	9.0	4.01	...	12.3	4.16	...	11.9	3.62	...	9.0
9	3.54	...	7.1	3.47	...	9.8	3.89	...	9.0	3.23	...	7.1	3.19	...	7.4	3.28	...	8.0	3.86	...	12.6	3.55	...	9.5	3.91	...	9.8	3.40	...	11.8	3.71	...	9.9	3.82	...	7.8			
10	2.98	1.8	5.3	2.74	1.6	9.9	3.16	2.2	8.1	2.93	2.4	7.4	3.11	3.0	6.5	3.22	2.7	8.3	3.63	3.7	10.3	3.66	3.8	9.9	3.34	4.1	11.0	3.23	1.7	6.3	3.28	3.7	7.6	3.08	2.3	7.2
11	2.63	...	6.9	2.68	...	8.2	2.58	...	6.0	2.19	...	7.4	2.56	...	6.0	2.19	...	7.4	3.47	...	9.7	3.45	...	7.8	3.02	...	8.2	3.01	...	9.0	3.42	...	9.0	3.62	...	9.0			
12	2.89	...	6.1	2.60	...	6.4	3.37	...	7.2	2.87	...	7.3	3.06	...	6.2	3.48	...	11.0	3.77	...	8.7	3.70	...	10.8	3.74	...	8.1	3.20	...	7.5	4.36	...	9.3	3.02	...	8.2			
13	4.27	...	8.5	4.16	...	9.0	5.92	...	14.5	5.04	...	23.4	6.17	...	12.5	5.06	...	18.0	10.34	...	22.9	8.42	...	15.4	8.42	...	15.7	7.46	...	19.4	6.62	...	17.6	5.22	...	16.9			
14	2.21	...	5.6	2.25	...	6.4	2.62	...	6.0	2.73	...	6.1	2.98	...	6.8	2.52	...	9.6	4.07	...	7.2	3.67	...	7.1	3.89	...	8.0	2.96	...	6.1	2.76	...	6.0	2.46	...	5.2			
15	1.82	1.0	7.0	1.62	1.6	4.2	2.05	2.0	4.1	2.13	1.3	5.8	2.98	1.9	9.6	3.47	3.1	9.0	3.80	3.3	8.4	3.40	3.5	9.6	3.60	2.8	9.3	2.32	1.8	6.2	2.32	1.8	6.2	1.90	1.4	4.8
16	2.24	...	5.0	2.21	...	4.3	2.55	...	8.4	2.29	...	5.1	2.91	...	6.5	2.91	...	7.0	3.46	...	8.0	3.53	...	7.0	3.01	...	8.0	2.53	...	6.1	2.53	...	6.1	2.34	...	5.0			
17	2.72	...	4.7	2.63	...	6.0	2.92	...	6.3	2.54	...	6.2	2.92	...	5.7	3.29	...	9.0	3.40	4.0	10.6	3.48	...	8.4	3.24	...	8.2	2.89	...	5.7	3.11	4.5	10.9	2.59	...	5.2			
18	1.90	...	4.0	1.79	...	4.0	2.27	...	5.2	2.77	...	5.3	2.31	...	7.2	2.83	...	6.8	3.48	...	11.1	3.42	...	7.8	3.28	...	8.4	2.89	...	12.8	2.28	...	6.0	2.01	...	4.0			
19	2.58	...	7.3	2.35	...	6.2	2.80	...	9.9	2.85	...	5.8	3.07	...	6.2	3.63	...	9.1	4.28	...	10.8	4.26	...	14.8	3.40	...	7.0	3.05	...	12.9	3.00	...	6.5	3.34	...	6.6			
20	3.31	...	7.9	3.00	...	8.7	3.63	...	9.5	3.19	...	7.8	3.72	...	8.9	3.09	...	10.0	4.32	...	16.0	4.24	...	11.7	3.48	...	11.4	3.86	...	15.5	3.53	...	8.1	3.86	...	8.8			
21	3.38	...	7.5	3.20	...	8.3	3.80	...	9.3	3.25	...	8.1	3.67	...	8.5	3.26	...	10.1	4.24	...	15.1	4.26	...	12.4	3.50	...	9.2	3.97	...	14.1	3.68	...	8.4	3.97	...	9.0			
22	3.10	...	7.8	3.11	...	9.3	3.50	...	8.0	3.48	...	10.2	3.37	...	7.6	3.29	...	10.4	3.56	...	10.2	4.08	...	12.8	3.31	...	11.0	3.46	...	7.9	3.69	...	8.0	3.53	...	8.6			
23	3.65	2.2	8.8	3.66	4.4	10.0	3.93	3.0	11.7	2.74	3.2	10.8	3.55	3.2	10.4	3.12	5.8	8.1	3.59	6.0	12.4	4.00	5.0	12.1	3.34	4.3	11.9	3.48	4.9	9.8	3.96	5.4	11.6	3.71	2.9	9.0
24	3.89	...	8.9	3.63	...	9.4	3.89	...	8.4	3.66	...	9.3	3.44	...	7.7	3.09	...	7.3	3.46	...	12.4	4.13	...	15.0	3.48	...	9.8	3.48	...	8.1	3.89	...	10.4	3.55	...	8.2			
25	3.59	...	8.1	3.53	...	9.2	3.46	...	7.0	3.45	...	9.7	3.25	...	7.7	3.09	...	7.3	3.44	...	11.7	4.44	...	7.8	3.49	...	9.8	3.46	...	8.1	3.89	...	10.4	3.55	...	8.2			
26	3.33	...	9.1	2.91	...	8.0	3.66	...	7.9	3.76	...	10.6	2.50	...	10.0	3.26	...	10.0	3.49	...	11.7	4.11	...	14.0	3.46	...	9.9	3.57	...	11.1	3.86	...	10.3	3.25	...	8.8			
27	3.84	...	7.8	3.76	...	8.6	4.16	...	10.9	3.84	...	10.2	3.54	...	8.1	3.39	...	11.2	3.89	...	13.3	4.22	...	12.4	3.68	...	9.8	3.76	...	10.1	4.03	...	8.6	3.65	...	8.6			
28	4.03	...	10.0	3.89	...	9.0	4.27	...	9.8	3.96	...	9.3	3.90	...	9.1	3.49	...	9.7	4.24	...	12.0	4.17	...	16.7	3.40	...	12.1	3.76	...	12.1	4.12	...	9.7	4.09	...	10.9			
29	2.79	...	6.7	2.77	...	7.1	2.73	...	7.1	2.73	...	7.1	3.13	...	7.9	2.71	...	7.3	3.24	...	11.7	3.43	...	8.3	3.53	...	8.0	3.58	...	10.0	3.26	...	7.5	3.26	...	7.5			
31	3.80	...	8.6	3.70	...	9.7	3.87	...	8.0	3.57	...	8.0	3.51	...	6.5	3.62	...	12.3	3.94	...	7.9	3.46	...	9.8	3.65	...	7.8	3.87	...	7.4	3.62	...	9.4	3.75	...	8.7			
32	4.12	2.1	8.8	3.81	2.4	11.8	4.13	1.6	9.7	3.80	2.2	9.4	3.64	2.5	10.6	3.22	3.0	9.6	3.45	3.5	8.1	3.02	3.8	8.6	3.34	4.8	9.8	3.57	3.1	9.3	3.82	2.3	9.1	3.97	2.7	9.4
33	3.56	...	9.0	3.63	...	9.5	4.20	...	11.2	3.52	...	8.74	...	9.0	3.28	...	7.4	3.04	...	9.2	3.72	...	10.0	4.35	...	13.3	3.33	...	8.0	3.67	...	9.4	3.78	...	9.1	3.76	...	8.1			
34	3.55	...	8.5	3.50	...	9.0	4.13	...	11.3	3.63	...	10.8	3.39	...	8.1	3.10	...	6.2	3.36	...	8.1	3.76	...	10.0	4.39	...	13.4	3.50	...	8.0	3.67	...	9.4	3.80	...	9.1	3.76	...	8.1			
35	3.96	...	9.2	3.88	...	9.0	4.42	...	9.6	3.52	...	12.4	3.79	...	7.7	3.38	...	11.4	4.11	...	13.4	4.50	...	10.2	3.58	...	11.6	3.97	...	15.5	3.42	...	8.1	3.91	...	10.5			
36	3.58	...	9.3	3.81	...	9.0	4.00	...	8.8	3.33	...	10.1	3.53	...	7.4	3.08	...	10.6	4.34	...	15.3	4.40	...	9.6	3.46	...	11.9	3.49	...	9.3	3.35	...	8.3	3.82	...	8.9			
37	3.59	...	9.3	3.58	...	9.0	4.09	...	9.1	3.53	...	11.0	3.53	...	7.4	3.13	...	10.6	4.16	...	14.0	4.12	...	9.6	3.49	...	11.9	3.49	...	9.3	3.35	...	8.3	3.82	...	8.9			
38	3.91	...	8.6	3.97	...	7.9	4.09	...	10.4	3.56	...	7.9	3.86	...	9.3	3.19	...	8.9	4.29	...	17.1	4.39	...	13.0	3.56	...	13.4	3.72	...	10.1	3.82	...	8.3	3.74	...	7.6			
39	3.58	...	6.1	3.70	...	7.8	3.74	...	7.9	3.29	...	7.0	3.16	...	9.1	3.21	...	7.9	4.33	...	9.6	4.46	...	10.4	3.40	...	14.5	3.57	...	11.5	3.12	...	9.8	3.41	...	8.7			
39-A	3.29	2.5	7.4	3.31	3.2	7.8	3.39	3.6	7.9	3.28	3.7	11.2	3.44	4.2	9.1	3.43	5.9	12.3	4.13	5.8	9.6	4.26	5.0	14.2	3.38	6.2	14.5	3.46	9.4	11.5	3.39	4.2	9.8	3.81	3.8	7.1
40	4.45	...	11.8	3.95	...	12.3	4.21	...	12.3	3.58	...	9.8	3.67	...	9.0	3.62	...	12.3	5.00	...	22.7	4.85	...	16.6	3.74	...	14.7	4.18	...	14.1	3.22	...	8.8	3.90	...	10.1			
41	4.24	...	9.1	3.92	...	9.0	4.20	...	12.3	3.68	...	10.3	4.37	...	11.7	3.84	...	13.5	4.56	...	22.7	4.85	...	16.6	3.74	...	14.7	4.18	...	14.1	3.22	...	8.8	3.90	...	10.1			
42	4.03	...	9.8	3.99	...	8.4	4.30	...	8.3	3.76	...	8.5	3.99	...	8.8	3.56	...	11.4	4.66	...	13.5	4.59	...	10.8	4.32	...	14.4	4.06	...	9.5	3.76	...	8.2	4.06	...	9.5			
43	2.54	1.4	7.3	2.42	1.9	5.2	2.74	2.3	7.4	2.70	2.0	7.0	3.35	2.2	9.5	3.32	2.9	9.7	3.91	3.1	9.4	3.73	4.2	10.6	3.32	3.3	9.9	2.32	4.7	12.8	2.95	5.7	7.3	3.50	4.3	6.3			
44	2.06	...	5.2	2.01	...	4.1	2.31	...	5.1	1.97	...	4.6	2.58	...	7.1	2.54	...	7.1	3.54	...	7.7	3.54	...	7.7	3.06	...	7.7	2.63	...	6.2	2.63	...	6.2	2.50	...	5.3			
45	2.73	1.2	6.1	2.42	2.1	5.4	2.50	1.2	4.7	2.26	1.6	3.9	3.32	2.4	7.2	3.06	3.5	6.8	3.60	3.2	8.2	3.40	2.9	8.9	3.62	2.7	9.1	3.10	2.4	5.9	3.11	1.9	6.5	2.50	1.5	6.3			
46	3.01	2.4	6.5	2.62	1.6	4.9	2.76	2.7	7.1	2.44	2.2	7.0	3.04	2.4	7.4	3.25	2.6	9.8	3.21	3.7	7.7	3.76	3.8	6.7	2.94	2.9	6.9	2.57	3.0	7.3	3.54	1.9	8.4	3.50	2.3	10.5			
47	2.54	2.0	6.1	2.32	1.5	4.7	2.72	2.4	6.8	2.08	1.6	4.7	2.72	2.4	6.4	3.00	2.6	8.2	3.03	3.0	7.2	3.23	2.8	8.1	2.73	2.4	6.9	2.57	2.6	6.4	3.01	1.9	8.4	3.50	2.3	10.5			
48	2.60	2.1	8.0	2.39	2.2	5.4	2.56	2.0	7.0	2.46	1.9	6.1	3.08	2.9	6.9	3.09	2.5	8.3	3.19	3.6	6.4	2.85	4.2	7.3	2.77	2.1	7.4	2.92	3.8	8.7	2.65	2.2	6.8	2.84	2.3	6.3			
49	2.96	...	7.1	2.77	...	6.3	3.01	...	7.0	2.61	...	6.1	3.08	...	7.4	3.08	...	8.3	3.31	...	6.4	2.98	...	7.2	2.83	...	7.1	2.83	...	6.7	3.40	1.7	8.4	3.46	2.3	6.3			
50	3.09	2.0	6.4	2.84	1.8	6.7	2.78	1.5	5.3	2.62	1.9	4.7	3.03	2.1	7.3	2.83	2.8	9.5	3.10	3.6	8.3	2.01	4.2	10.6	3.17	2.4	7.7	3.40	3.1	7.8	3.13	2.6	6.7	3.37	2.6	6.5			
51	3.24	...	7.1	3.92	...	5.6	2.54	...	6.5	3.07	...	5.3	3.44	...	6.8	3.74	...	7.8	4.78	...	11.2	3.82	...	9.5	3.59	...	8.8	2.66	...	7.8	3.52	...	7.3	3.94	...	7.0			
52	2.06	...	4.9	2.34	...	4.4	2.54	...	4.4	2.36	...	4.8	2.42	...	6.4	2.42	...	6.4	3.42	...	6.4	3.54	...	7.6	2.91	...	7.3	3.26	...	7.0	2.73	...	6.4	2.73	...	6.4			
53	2.08	2.0	5.1	2.08	1.6	4.4	2.36	2.0	4.8	2.42	2.8	6.4	3.42	3.6	6.7	3.54	3.2	7.6	3.91	3.3	8.4	3.05	4.2	7.6	2.91	2.3	7.3	3.26	2.9	7.0	2.73	2.4	6.4	2.73	2.4	6.4			
54	2.27	...	5.9	2.02	...	6.1	2.89	...	6.0	2.79	...	7.1	3.26	...	7.8	4.13	...	10.0	4.01	...	9.2	4.37	...	9.7	3.21	...	8.1	3.44	...	8.3	3.02	...	8.4	2.86	...	6.4			
55	2.32	1.9	5.2	2.32	1.9	6.1	2.74	1.9	6.7	2.63	2.2	7.7	3.23	3.0	7.7	3.19	4.5	7.7	3.34	2.4	8.1	2.63	3.2	7.7	3.43	2.8	8.3	3.02	2.5	7.4	2.68	2.4	6.4	2.68	2.4	6.4			
56	2.70	1.7	4.7	2.88	3.6	6.3	3.22	2.7	5.7	2.79	1.8	5.2	2.98	1.7	5.6	3.61	3.0	7.7	3.64	3.0	6.8	3.61	2.9	8.2	3.29	3.1	7.3	3.09	3.7	9.3	2.49	3.2	5.6	2.96	2.1	5.6			
57	3.41	3.3	6.7	3.38	3.9	6.9	3.45	2.8	9.1	2.91	2.8	9.6	3.20	3.2	9.5	3.30	3.4	8.0	4.33	3.2	10.3	4.61	5.0	12.1	3.38	5.0	12.1	3.10	3.7	6.7	3.06	2.6	7.3	3.04	2.6	7.8			
58	2.79	...	5.8	2.77	...	6.1	2.98	...	6.1	2.66	...	6.1	3.06	...	7.4	3.08	...	8.3	3.66	...	7.4	3.48	...	8.3	3.48	...	12.1	3.09	...	6.9	3.46	...	7.4	3.46	...	7.4			
59	3.54	2.9	7.3	3.90	3.9	7.3	4.07	2.9	8.3	3.78	4.2	8.6	4.45	5.0	13.8	4.19	6.1	9.2	4.63	5.4	11.2	4.63	5.4	11.2	4.23	5.9	11.2	3.74	5.8	8.7	3.63	3.8	9.3	4.09	3.8	9.6			
60	3.50	2.8	8.5	3.50	3.4	6.7	3.52	3.4	8.2	3.34	3.0	8.0	3.49	3.7	9.2	3.23	5.0	8.0	3.73	3.5	9.2	3.73	3.5	9.2	3.37	3.5	9.2	3.37	3.5	9.2	3.37	3.5	9.2	3.37	3.5	9.2			
61	2.59	2.2	6.3	3.47	3.0	6.4	3.84	2.5	6.6	3.37	2.2	6.3	4.03	3.0	10.6	4.00	2.2	10.0	4.27	6.2	11.4	4.61	4.8	12.9	3.18	2.0	10.2	3.52	3.2	8.2	2.89	2.4	10.0	3.62	1.8	7.5			
62	3.07	2.3	10.1	2.82	2.6	5.5	3.04	2.3	5.8	3.10	2.6	6.9	3.76	2.3	10.0	3.91	1.9	11.2	3.79	2.3	13.4	4.18	4.7	12.2	3.36	3.1	12.4	2.97	3.3	6.4	2.25	1.9	6.6	2.14	1.5	9.0			
63	2.50	1.4	6.2	2.11	1.6	4.6	2.70	1.1	5.8	2.87	2.6	6.4	4.02	2.9	9.3	3.92	2.0	9.9	3.27	2.6	9.0	3.39	4.3	8.4	2.76	2.6	6.6	2.49	2.3	7.1	2.91	1.6	6.4	2.35	1.6	6.4			
64	2.85	1.7	7.8	3.16	1.7	7.7	3.61	...	6.7	3.32	...	8.9	3.88	...	7.8	4.26	...	7.1	4.18	...	8.1	3.72	...	8.9	3.87	...	8.7	3.89	...	7.7	3.88	...	6.5	3.26	...	6.5			
65	2.62	1.9	7.7	2.89	1.8	9.2	3.49	2.2	7.6	3.03	2.8	8.5	3.49	1.9	10.0	4.77	2.8	9.3	4.48	2.1	12.4	3.80	2.2	7.4	3.49	2.5	10.2	3.11	1.9	8.9	3.01	1.7	6.5	2.32	1.9	5.9			
66	2.62	1.9	7.7	2.89	1.8	9.2	3.49	2.2	7.6	3.03	2.8	8.5	3.49	1.9	10.0	4.77	2.8	9.3	4.48	2.1	12.4	3.80	2.2	7.4	3.49	2.5	10.2	3.11	1.9	8.9	3.01	1.7	6.5	2.32	1.9	5.9			
67	2.62	1.9	7.7	2.89	1.8	9.2	3.49	2.2	7.6	3.03	2.8	8.5	3.49	1.9	10.0	4.77	2.8	9.3	4.48	2.1	12.4	3.80	2.2	7.4	3.49	2.5	10.2	3.11	1.9	8.9	3.01	1.7	6.5	2.32	1.9	5.9			
68	2.73	2.3	6.2	2.57	2.0	6.5	3.04	2.0	6.8	2.92	2.6	8.1	3.25	3.0	6.4	3.63	2.2	6.9	4.23	3.8	9.5	3.23	3.8	7.3	2.62	4.1	7.4	2.52	2.0	6.2	2.84	2.1	6.1	2.81	2.4	6.3			
69	2.73	2.3	6.2	2.57	2.0	6.5	3.04	2.0	6.8	2.92	2.6	8.1	3.25	3.0	6.4	3.63	2.2	6.9	4.23	3.8	9.5	3.23	3.8	7.3	2.62	4.1	7.4	2.52	2.0	6.2	2.84	2.1	6.1	2.81	2.4	6.3			
70	4.76	2.1	8.9	3.98	1.8	7.7	4.34	2.7	8.8	4.64	2.1	10.0	5.02	2.3	14.2	5.14	2.6	10.3	4.44	2.5	10.6	4.72	4.4	11.2	3.67	3.2	11.4	3.12	1.8	8.0	3.46	2.0	8.4	3.95	2.1	7.8			
71	3.78	...	7.7	3.03	...	7.1	3.68	...	7.2	3.29	...	7.2	3.57	...	8.1	4.66	...	7.2	4.98	...	12.3	4.24	...	10.1	3.08	...	7.7	3.09	...	8.2	2.92	...	7.3	3.14	...	6.7			
72	3.60	...	7.5	3.03	...	6.1	3.68	...	6.8	3.02	...	6.4	3.57	...	8.1	4.66	...	7.2	4.98	...	12.3	4.24	...	10.1	3.08	...	7.7	3.09	...	8.2	2.92	...	7.3	3.14	...	6.7			
73	4.19	...	7.5	3.78	...	6.6	4.38	...	10.0	3.94	...	7.8	4.21	...	9.1	4.86	...	15.1	3.99	...	9.8	3.90	...	5.8	2.98	...	7.2	2.20	...	9.7	2.82	...	7.8			
74	4.31	1.8	8.9	3.14	1.6	5.7	4.10	2.4	7.1	3.84	2.2	7.9	4.15	2.7	9.9	5.32	2.9	8.0	5.30	4.3	11.1	4.10	4.3	10.6	3.60	2.7	9.1	3.07	2.6	6.8	2.77	2.1	6.8	2.78	2.1	6.8			
75	3.22	3.0	7.1	2.97	2.3	6.8	3.42	2.8	8.8	3.21	3.2	9.1	3.61	3.5	10.7	3.80	4.2	10.9	4.38	5.8	10.4	4.15	5.1	10.4	3.43	4.2	10.6	3.23	3.7	7.6	3.02	2.7	7.6	3.02	2.7	7.6			
76	3.40	3.0	7.1	3.03	2.3	6.8	3.42	2.8	8.8	3.21	3.2	9.1	3.61	3

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record. For day or less.	Years of Record. For month and year.	Maximum Observed Precipitation. Last year included.	Maximum Observed Precipitation. Number of Minutes. 5.	15.	60.	Maximum Observed Precipitation. Day of 24 hours.	Maximum Observed Precipitation. Month.	Annual Precipitation. Minimum.	Annual Precipitation. Mean.	Annual Precipitation. Maximum.	Observed Precipitation, in Percentages of Mean Annual Rainfall. Mean daily interval between rainfall.	Maximum During Short Periods. Number of Minutes. 5.	15.	60.	Mean for month of maximum record.	Maximum. In 24 hours.	Maximum. In one calendar month.	Annual. Minimum.	Annual. Mean.	Annual. Maximum.	Main references.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
119*	New Orleans, La.	51	57	81	1927				14.0	22.7	31.1	56.8	85.7	0.8				10.1	24.6	49.0	57	100	151	1; 2
120	Shreveport, La.	249	56	56	1927	0.59	1.20	3.15	5.6	16.5	23.1	44.3	66.6	1.0	1.3	2.7	7.1	6.9	19.4	37.3	52	100	150	1; 2
121	Dallas, Tex.	466	16	51	1927	0.81	1.62	2.33	6.1	13.7	18.0	37.3	59.5	1.1	2.2	4.3	6.2	8.3	19.4	36.9	48	100	160	2; 1
122	Fort Worth, Tex.	670	30	30	1927	0.56	1.40	2.75	5.8	17.6	17.9	31.6	49.4	1.3	1.8	4.4	5.7	13.0	27.9	55.7	57	100	153	1; 2
123	Palestine, Tex.	510	41	41	1927	0.59	1.56	4.46	6.3	17.2	24.0	40.7	61.2	1.1	1.4	3.8	10.9	11.1	15.5	42.3	59	100	150	2
124	Taylor, Tex.	583	23	25	1923	0.90	2.53	4.25	23.1	25.9	14.6	31.9	51.5	1.1	1.8	7.9	13.3	10.6	72.4	81.2	46	100	161	
125	Austin, Tex.	541	40	50	1929					19.6	15.4	32.2	64.7	1.0				10.6		59.6	47	100	154	2
126	Houston, Tex.	138	32	45	1926				7.5	19.2	30.6	47.4	72.3					10.4	15.8	40.5	68	100	154	1
127	Galveston, Tex.	59	50	50	1920			5.32	14.4	20.4	21.4	47.1	78.4	1.0			11.3	11.5	30.6	55.3	43	100	167	1
128	San Antonio, Tex.	701	36	50	1920				7.1	11.7	16.1	27.1	50.3	1.5				11.2	20.6	46.4	38	100	152	2
129	Corpus Christi, Tex.	20	41	41	1927				8.3	13.6	5.4	24.9	48.2	1.2				16.4	33.3	55.5	22	100	194	
130	Brownsville, Tex.	58	41	50	1920					20.6	9.3	26.4	60.1	1.0				16.4		116.0	42	100	168	
131	Eagle Pass, Tex.	800		45	1920					13.5	7.0	20.8	44.4	2.3				12.4		65.0	34	100	213	2
132	El Paso, Tex.	3 762	48	48	1927				6.5	8.4	2.0	9.0	19.4	2.0				21.6	70.7	100.0	24	100	209	1; 2
133	Abilene, Tex.	1 738	41	41	1926	0.67	1.51	3.47	6.8	15.7	10.9	24.4	41.9	0.7	2.7	6.2	14.2	9.6	27.9	64.4	45	100	171	1; 2
134	Amarillo, Tex.	3 676	36	46	1927	0.53	1.19	1.93	4.0	9.6	11.1	22.0	39.7	1.4	2.4	5.4	8.9	13.7	18.2	44.5	50	100	181	1; 2
135	Oklahoma, Okla.	1 247		30	1920					12.3	15.7	31.0	52.0	1.2				11.3		59.6	51	100	166	2
136	Fort Gibson, Okla.	556		50	1920					13.1	16.8	37.2	59.5	1.5				9.6		55.2	52	100	168	2
137	Bentonville, Ark.	1 303	17	17	1923	0.75	1.58	2.23	5.1	11.2	34.6	41.4	56.3	0.9	1.8	3.8	5.4	6.2	12.3	29.5	84	100	136	2
138	Fort Smith, Ark.	457	41	41	1922	0.57	1.14	2.06	5.1	15.0	19.8	38.3	64.6	1.1	1.5	3.0	6.4	8.8	13.3	39.3	54	100	169	2
139	Little Rock, Ark.	257	51	49	1927	0.90	1.52	2.42	9.6	20.6	30.0	48.2	70.5	0.9	1.2	4.2	8.0	9.5	19.3	39.0	66	100	145	2
140	Memphis, Tenn.	409	57	57	1927	0.78	1.08	1.97	9.7	18.2	33.4	48.5	73.5	0.9	1.6	2.2	4.1	7.9	20.0	37.5	69	100	162	1
141	Nashville, Tenn.	624	18	57	1927	0.75	1.20	2.09	6.0	14.5	32.7	47.2	67.2	0.9	1.6	2.7	4.4	9.9	12.7	30.7	69	100	142	1
142	Chattanooga, Tenn.	742	18	42	1920	0.62	1.20	2.07	4.0	10.0	30.2	49.2	68.1	0.9	1.2	2.6	4.0	9.3	14.8	29.4	64	100	158	2
143	Knoxville, Tenn.	996	50	53	1920	0.57	1.24	2.04	6.2	17.3	25.1	48.5	73.9	0.7	1.2	2.6	6.0	9.3	14.8	34.0	64	100	158	2
144	Lexington, Ky.	989	39	62	1920	0.34	0.91	2.37	5.4	11.1	28.7	44.5	63.3	0.7	0.8	2.0	5.3	8.7	12.1	25.0	64	100	142	2
145	Louisville, Ky.	525	50	57	1927	0.55	1.20	1.59	5.5	16.5	29.2	43.7	56.5	0.5	1.1	2.7	3.6	8.9	12.0	37.8	67	100	129	1
146	Evansville, Ind.	431	31	51	1927	0.53	1.19	2.79	6.9	14.6	29.0	43.0	70.6	0.9	1.2	2.8	6.5	7.4	16.1	34.0	67	100	164	1; 2
147	Cairo, Ill.	358	50	57	1927	0.34	0.86	3.15	5.7	15.0	26.5	41.3	64.1	0.9	0.8	2.1	7.6	9.4	13.8	36.4	64	100	156	1; 2
148	St. Louis, Mo.	567	57	91	1927	0.56	1.07	2.95	7.0	17.1	23.4	39.7	63.8	0.9	1.4	2.7	7.4	11.4	17.6	43.0	59	100	173	1; 2
149	Hannibal, Mo.	534	25	43	1922	0.59	1.39	2.09	5.0	15.0	25.1	39.2	57.1	0.9	1.3	3.0	4.2	7.5	12.2	27.6	57	100	145	2
150	Columbia, Mo.	784	20	37	1922	0.70	1.22	2.73	6.0	11.5	21.3	37.6	54.6	0.9	1.9	3.2	7.3	12.2	17.6	36.8	57	100	145	2
151	Springfield, Mo.	1 302	40	50	1927				6.5	12.5	25.2	42.2	65.7					11.6	15.6	36.0	62	100	155	2; 1
152	Kansas City, Mo.	963	33	50	1922	0.57	1.15	4.74	7.0	16.2	19.0	36.8	49.5	1.0	1.5	3.1	12.9	11.5	15.5	44.4	57	100	158	2
153	Oregon, Mo.	1 048	30	68	1922				7.0	15.0	21.4	35.6	49.7	0.9				13.6	19.7	42.1	60	100	140	2
154	Leavenworth, Kans.	913		57	1922					15.8	14.6	34.4	59.6					14.5		46.0	42	100	173	2
155	Manhattan, Kans.	1 081	45	65	1922				5.0	12.0	16.4	30.4	51.4					24.4	48.6	71.4	46	100	164	2
156	Dodge City, Kans.	2 509	48	49	1922	0.55	1.25	3.47	6.0	12.8	10.1	20.1	33.5	1.3	2.7	6.2	17.3	14.4	29.8	63.7	50	100	168	2
157	Wallace, Kans.	3 303	39	39	1922				6.1	12.6	7.4	16.2	30.0					19.4	37.6	67.8	46	100	210	2
158	North Platte, Nebr.	2 821		52	1925					8.5	11.2	18.4	32.7	1.2				14.8		46.2	61	100	178	4
159	Lincoln, Nebr.	1 189	30	51	1920				8.4	14.2	15.1	26.1	41.2	1.0				12.9	29.9	50.8	54	100	147	1
160	Omaha, Nebr.	1 160	57	57	1927	0.67	1.27	2.53	7.0	12.7	15.5	28.5	48.0	1.0	2.3	4.5	8.9	16.6	24.6	44.6	54	100	172	1; 2
161	Des Moines, Iowa.	861	42	43	1920	0.66	1.36	2.65	5.1	15.8	18.2	34.6	54.8	0.9	2.1	4.2	8.3	14.6	15.9	49.2	57	100	177	2
161-A	Cedar Rapids, Iowa.	733	39	39	1920				7.8	15.4	18.7	31.1	44.9					10.8	25.1	49.5	60	100	146	
162	Dubuque, Iowa.	699	18	65	1920	0.80	1.52	2.70	5.6	14.2	19.3	33.4	55.1	0.9	2.4	4.6	8.1	13.2	16.2	42.5	58	100	165	2
163	Davenport, Iowa.	590	55	55	1920	0.54	1.36	2.70	5.2	15.0	17.7	32.4	51.8	0.9	1.7	4.2	8.0	13.2	16.2	42.3	58	100	163	1; 2
164	Keokuk, Iowa.	614	15	50	1920	0.56	1.18	2.38	4.8	12.7	21.8	33.6	51.4	1.0	1.7	3.5	7.1	11.4	14.3	37.8	65	106	154	2
165	Springfield, Ill.	644	48	49	1927	0.66	1.41	2.75	5.8	15.2	22.8	36.4	58.2	0.9	1.8	3.9	7.6	10.1	16.2	41.8	62	100	160	1; 2
166	Peoria, Ill.	609	72	72	1927	0.67	1.40	2.64	6.0	15.0	24.5	36.4	51.3	0.9	1.4	3.0	6.7	9.3	14.4	34.0	57	100	158	2
167	Chicago, Ill.	524	50	84	1927	0.42	0.90	1.53	6.2	11.3	22.4	33.1	47.1	0.8	1.3	2.7	4.6	9.5	18.7	34.1	68	100	142	1; 2
168	Fort Wayne, Ind.	856	24	52	1926	0.81	1.27	2.37	4.9	10.6	26.7	37.1	47.0	....	2.2	3.4	6.1	10.9	13.1	28.6	72	100	127	2
169	Indianapolis, Ind.	822	57	57	1927	0.83	1.30	2.66	6.8	13.3	26.3	40.4	57.6	0.8	2.0	3.2	6.6	9.7	16.8	32.5	75	100	143	1; 2
170	Terre Haute, Ind.	576	42	54	1928	1.15	1.35	2.57	5.9	11.4	25.3	38.2	48.3	0.9	2.0	3.6	6.7	9.4	13.5	30.8	67	100	139	2
171	Cincinnati, Ohio.	628	51	93	1927	0.78	1.78	2.69	5.2	11.7	18.0	40.7	65.2	0.8	1.9	4.4	6.4	8.9	12.8	28.7	44	100	160	2; 1
172	Portsmouth, Ohio.	527		92	1921				3.5	11.4	24.3	41.2	57.6	0.9				9.2	8.5	27.0	59	100	140	
173	Marietta, Ohio.	627	28	104	1923				3.6	12.4	27.6	41.1	57.1	0.9				11.2	10.4	30.2	58	100	140	
174	Columbus, Ohio.	918	44	50	1927	0.38	1.00	2.86	3.8	9.6	26.5	36.4	51.3	0.7	1.0	2.7	7.1	9.9	10.4	26.4	72	100	141	1
175	Cleveland, Ohio.	762	51	56	1927	0.44	1.27	1.86	3.6	9.6	26.5	36.4	51.3	0.7	1.3	3.7	5.6	9.9	14.6	27.7	72	100	141	1; 2
176	Sandusky, Ohio.	629	44	61	1927	0.56	1.34	2.22	4.2	10.6	23.5	33.1	46.3	0.7	1.7	4.0	6.7	8.9	12.7	32.0	71	100	140	2
177	Toledo, Ohio.	709	51	60	1920	0.60	1.77	3.58	6.0	10.2	21.3	31.7	45.9	0.8	1.9	5.6	11.3	8.0	16.9	32.1	67	100	145	2
178	Detroit, Mich.	730	57	57	1927				4.7	8.8	21.1	32.0	47.7	0.7				10.4	16.7	27.5	66	100	149	1
179	Lansing, Mich.	893		85	1917					9.3	18.2	30.6	46.4	0.7				11.7		27.0	57	100	143	1
180	Alpena, Mich.	909	56	56	1927				3.9	13.2	17.4	30.7	45.6	0.7				9.8	12.7	43.0	57	100	148	1
181	Marquette, Mich.	734		59	1925					7.3	23.3	34.4	49.3	0.8				10.1		37.2	57	100	143	1
182	Milwaukee, Wis.	681	57	57	1927				5.8	10.0	18.7	30.8	50.4	0.8				11.3	18.8	32.5	61	100	164	1
183	Madison, Wis.	974	61	61	1920				5.2	10.7	13.5	31.2	52.9	0.9				10.2	16.7	34.3	43	100	169	2
184	La Crosse, Wis.	714		60	1920	0.31	0.87	1.85	3.2	8.3	23.3	34.4	49.3	0.9	1.0	2.9	5.8	7.8	23.2	38.9	54	100	144	2
185	St. Paul, Minn.	940	56	80	1926	0.40	1.00	2.28	5.7	11.7	10.2	27.2	49.7	0.9	1.5	3.7	8.1	15.2	20.9	43.0	57	100	183	1; 2
186	Duluth, Minn.	1 133	56	57	1927	0.41	1.06	2.26	5.3	13.3	18.1	28.5	45.3	0.8	1.5	3.7	6.1	12.6	20.9	40.2	61	100	163	2
187	Moorhead, Minn.	935	46	46	1926	0.53	1.12	2.21	5.2	9.6	9.9	23.1	34.0	0.9	2.3	4.4	9.6	12.4	22.5	41.6	43	100	147	1
188	Bismarck, N. Dak.	1 674		51	1925					9.9	11.0	17.0	30.9	1.1				20.3		58.2	65	100	182	2
189	Williston, N. Dak.	1 873	57	57	1927					9.1	9.0	15.0	30.1	1.1				20.3		53.3	62	100	144	2
190	Huron, S. Dak.	1 306	39	40	1920				3.9	11.6	10.2	20.9	30.1	1.0				16.4	18.9	55.4	49	100	144	2
191	Pierre, S. Dak.	1 572	29	30	1920				2.7	6.1	7.8	17.0	23.6	1.2				16.5	21.8	35.9	46	100	129	2
192	Rapid City, S. Dak.	3 253	32	33	1920				2.6	6.0	5.1	16.8	30.2	1.1				12.6	27.1	54.8	54	100	153	2
193	Sheridan, Wyo.	3 790	15	28	1920	0.49	0.85	1.32	2.1	6.6	8.0	14.4	22.5	0.9	2.4	5.9	9.2	16.6	14.6	45.8	56	100	156	2
194	Cheyenne, Wyo.	6 088	50	50	1920	0.30	0.88	1.18	1.4	7.7	5.0	14.0	22.7	0.8	2.1	4.8	7.1	12.9	15.4	55.0	54	100	159	2
195	Yellowstone Park, Wyo.	6 200	32	40	1927	0.23	0.35	0.63	2.8	8.9	7.9	18.0	28.6	0.8	1.3	1.8	3.5	8.2	12.8	49.4	44	100	159	1; 2
196	Miles City, Mont.	2 371	31	42	1920				3.4	5.9	9.1	13.5	22.7	1.2				11.7	25.2	43.7	67	100	168	2
197	Havre, Mont.	2 503	19	41	1927	0.43	0.77	1.60	2.2	5.8	7.2	13.5	22.7	1.2	2.2	3.2	5.7	11.9	13.5	45.8	50	100	161	2
198	Kalispell, Mont.	2 973	24	26	1921	0.43	0.90	1.03	1.8	3.2	10.7	15.2	19.7	0.8	2.8	5.9	6.8	10.1	11.8	24.2	70	100	130	2
199	Helena, Mont.	4 110	58	58	1920	0.24	0.49	1.04	1.2	3.4	6.1	13.4	20.0	1.0	1.6	3.7	7.8	9.9	11.0	27.6	60	100	149	2
200	Pocatello, Idaho.	4 477			1920				1.6	4.3	9.3	13.9	22.4	1.1				11.3	11.5	30.9	67	100	161	2
201	Boise, Idaho.	2 739	46	64	1927				2.7	7.7	6.0	12.5	25.8	1.1				11.6	20.0	57.0	44	100	191	2
202	Lewiston, Idaho.	737		61	1920				1.9	6.3	9.1	12.4	21.7	0.9				10.5	14.2	47.0	68	100	163	2
203	Spokane, Wash.	1 943	19	46	1926	0.29	0.38	0.77	2.2	4.8	7.9	16.3	26.0	0.9	1.9	2.3	4.7	12.9	13.5	35.6	61	100	159	1; 2
204	Seattle, Wash.	346	34	57	1927	0.20	0.52	0.85	1.1	3.3	21.7	30.6	48.4	0.7	0.9	1.6	2.4	10.6	9.7	30.0	43	100	168	1; 2
205	Forks (N. W.), Wash.	480		15	1921					3.1	30.6	122.3	152.8	0.6				14.8		25.6	69	100	128	2
206	North Head, Wash.	211	19	44	1926	0.27	0.37	0.88	4.8	17.3	30.9	55.5	78.8	1.5	0.6	0.7	1.6	16.1	7.7	25.6	89	100	158	2
207	Astoria, Ore.	24		68	1922					24.7	49.4	77.0	101.6	0.5				15.7		32.1	64	100	132	2
208	Portland, Ore.	57	57	57	1927	0.40	0.83	1.31	7.7	20.1	30.6	42.8	67.2	0.6	0.9	1.9	3.1	15.4	18.0	47.0	72	100	157	1
209	Canada Locke, Ore.	100			1920					24.0	36.4	62.4	105.2	0.6				16.7		44.2	64	100	128	2
210	The Dalles, Ore.	66		65	1922					10.8	7.5	16.3	42.6					16.9		44.8	64	100	128	2
211	Baker, Ore.	3 471	31	31	1920				1.8	4.0	4.1	12.3	18.3	1.0				17.9	14.8	32.5	63	100	148	2
212	Roseburg, Ore.	510	45	45	1920	0.55	1.10	1.14	4.1	12.5	21.7	41.2	63.5	0.8	1.7	3.3	3.4	10.7	17.8	42.5	61	100	161	2
213	Monumental, N. W. Calif.	2 700±		6	1910					43.8	69.3	109.1	153.5						19.4		40.0	63	100	140	2
214	Crescent City, Calif.	125		34	1923					21.9	47.5	74.8	107.6	0.9					13.5		42.6	63	100	144	2
215	Fort Gaston, Calif.	397		37	1923					19.8	34.6	56.0	87.1						16.2		42.6	63	100	144	2
216	Eureka, N. W. Calif.	64	37	37	1923				5.1	19.5	22.4	41.8	64.5	0.9					16.3	12.3	46.6	54	100	154	2
217	Weaverville, Calif.	2 162		56	1923					7.9	14.2	27.6	43.5						12.3		46.6	54	100	154	2
218	Delta, Calif.	84		34	1916				5.3	3.3	28.1	65.0	114.8						19.6		32.2	43	100	177	2
219	Red Bluff, Calif.	307	47	48	1923	0.26	0.57	1.72	6.1	20.7	12.9	24.6	49.0	1.4	1.1	2.5	7.0	19.2	24.7	84.0	52	100	199	4; 2
220	Marysville, Calif.	67		62	1922					14.4	10.8	19.6	39.7						20.2		53.5	58	100	187	2
221	Helen Mine, Calif.	2 750		32	1922					17.1	15.4	34.0	137.5						20.2		53.5	58	100	187	2
222	Santa Rosa, Calif.	181		35	1923					18.4	13.6	29.9	46.5	1.4					22.4		65.0	46	100	156	2
223	Port Rose, Calif.	100		49	1922					9.2	9.2	19.4	29.6	1.4					16.4		65.0	46	100	156	2
224	Point Reyes, Calif.	490	32	43	1923	0.20	0.60	1.40	3.1	11.4	7.3	20.6	32.4	1.3	1.0	3.4	6.8	16.6	18.0	55.2	36	100	157	2
225	Mt. Tamalpais, Calif.	2 375	23	26	1923				3.0	15.6	18.5	27.5	42.3	1.2					21.2	10.9	56.6	49	100	153	2
226	San Francisco, Calif.	206	57	57	1923				4.7	12.4	13.4	22.5	38.2	1.4					21.2	20.8	56.6	49	100	153	2
227	San Jose, Calif.	95	17	49	1923				4.6	12.4	6.6	15.1	28.5	1.6					20.3	30.4	82.0	44	100	169	2
228	Gilroy, Calif.	470		31	1919					11.4	7.2	16.7	29.0						20.8		70.0	43	100	169	2
229	Monterey, Calif.	10		51	1915					11.7	7.2	16.7	29.0						20.8		70.0	43	100	169	2
230	San Luis Obispo, Calif.	201	28	56	1926				6.0	18.2	6.9	23.1	50.0	1.9	1.2	1.9	5.8	23.3	28.4	96.0	33	100	237	2; 4
231	Santa Barbara, Calif.	150		56	1922					17.2	7.2	19.7	43.2	2.3					23.8		92.0	39	100	231	2
232	Los Angeles, Calif.	381	47	51	1927				5.1	16.4	6.8	15.2	40.8	2.6					17.4	32.5	104.0	22	100	231	1; 2
233	Lowe Observatory, Calif.	3 420		24	1919					19.9	8.3	26.7	58.5						22.7		74.6	31	100	219	2
234	Mount Wilson, Calif.	5 850		24	1919					24.0	8.4	29.5	64.0						25.3		72.4	26	100	156	2
235	Riverside, Calif.	851		40	1919					9.7	3.9	10.7	25.2	2.6					21.7		90.6	36	100	236	2
236	San Diego, Calif.	87	56	79	1927				3.2	9.3	4.3	9.8	27.6	3.3					19.2	32.6	95.0	44	100	283	1; 4
237	Campo, Calif.	2 543		31	1919			8.6		20.4	11.3	19.8	35.1				45.6	18.7		103.0	57	100	178	2
238	Cuyamaca, Calif.	4 677		33	1919					36.0	15.2	32.2	68.1	1.7				19.0		98.0	49	100	170	2
239	Sterling, Calif.	255		43	1919				5.4	Tr.	2.2	7.4						19.0		245.0	0	100	336	2
240	Indio, Calif.	−20		43	1919				6.0	0.1	1.4	5.1		8.3				20.3		214.4	0	100	284	2

(Continued.)

Station No.	January. Mean month.	January. Maximum. In 24 hours.	January. Maximum. Month.
(1)	(26)	(27)	(28)
119	4.54	4.21	12
120	3.97	5.12	2
121	2.58	2.7	8
122	1.51	3.3	4
123	3.29	4.8	11
124	1.84	1.9	7
125	1.97	....	8
126	3.59	4.5	13
127	2.62	5.4	10
128	1.59	2.5	1
129	1.02	2.1	5
130	1.72	4.0	6
131	1.25	....	4
132	0.46	1.3	1
133	0.48	1.3	1
134	0.56	1.3	2
135	1.17	....	4
136	2.27	....	1
137	2.68	3.5	11
138	3.23	3.9	3
139	4.06	5.8	11
140	4.58	4.0	12

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.	
		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
119	4.54	4.24	11.1	4.17	5.77	13.8	4.64	8.75	15.9	4.82	4.04	14.9	4.42	9.13	5.7	5.47	5.52	14.6	6.54	7.51	5.5	5.72	5.92	22.7	4.61	8.23	12.9	3.47	7.81	13.4	3.68	4.49	8.3	4.67	4.82	14.4
120	3.97	7.52	12.1	3.46	5.03	8.7	4.32	4.55	11.7	4.75	4.66	11.0	4.24	8.39	14.5	3.45	8.0	8.3	3.48	8.63	5.2	4.04	7.07	16.5	3.04	7.01	16.5	2.68	4.21	11.7	2.64	5.23	9.5	4.45	7.21	15.5
121	2.58	2.7	8.5	2.35	1.4	10.7	3.37	6.4	9.5	4.26	6.1	13.0	4.27	4.42	12.9	3.89	4.3	10.2	2.89	2.81	9.2	3.08	5.2	13.7	2.72	2.5	7.6	2.73	3.7	10.9	2.76	4.6	10.1	2.84	2.5	5.4
122	1.51	3.3	4.7	1.52	2.3	4.6	2.18	4.3	7.0	4.12	5.8	17.0	4.30	7.0	10.7	3.08	2.9	8.7	2.57	5.21	6.3	2.72	5.4	10.5	2.46	7.4	9.1	2.69	4.0	9.4	2.67	4.7	7.8	1.84	2.5	5.4
123	3.29	4.8	11.1	2.98	3.7	7.9	3.36	4.7	9.5	4.27	4.6	11.2	4.52	5.77	7.3	3.43	4.7	9.8	2.75	6.31	0.0	2.44	5.2	9.8	2.46	3.1	25.9	2.97	7.5	9.8	2.41	3.2	8.2	2.42	8.6	11.7
124	1.84	1.9	7.7	1.74	1.1	7.1	1.95	2.5	6.1	3.99	3.8	16.2	4.16	4.7	9.5	2.66	4.3	8.6	2.24	6.5	14.4	2.07	4.8	9.1	2.46	3.3	12.8	3.06	4.2	12.5	2.55	3.6	8.0	2.72	3.6	11.1
125	1.97	3.8	0.0	2.33	3.9	9.4	2.21	4.7	7.2	3.81	4.9	19.8	4.28	4.8	14.1	2.48	3.4	8.4	2.38	5.24	12.6	2.40	4.0	10.9	2.45	3.2	12.4	3.05	4.3	9.9	2.57	3.2	10.3	2.64	7.6	14.4
126	3.50	6.51	13.1	3.04	3.9	9.0	3.48	4.4	10.7	3.78	4.7	10.6	4.93	5.42	19.3	4.47	4.21	9.5	3.88	4.74	8.8	4.12	6.37	13.4	4.15	8.11	13.4	3.70	5.3	9.9	3.57	5.2	10.3	3.67	5.4	9.4
127	3.62	5.4	10.4	3.10	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
128	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
129	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
130	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
131	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
132	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
133	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
134	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
135	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
136	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
137	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
138	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
139	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
140	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
141	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
142	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
143	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
144	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
145	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
146	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
147	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
148	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
149	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
150	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
151	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
152	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
153	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
154	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
155	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
156	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
157	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
158	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
159	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
160	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
161	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
162	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
163	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
164	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
165	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
166	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
167	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
168	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
169	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
170	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
171	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
172	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
173	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
174	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
175	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
176	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
177	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
178	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
179	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
180	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
181	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
182	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
183	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
184	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
185	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
186	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
187	3.03	5.4	10.4	3.03	3.8	8.3	2.90	4.0	8.1	3.23	9.2	11.0	3.23	8.10	7.3	4.75	8.0	31.5	3.98	16.4	18.7	3.98	5.0	19.5	4.41	7.0	26.0	4.02	6.0	9.2	4.18	24.1	37.9	4.02	6.0	9.2
188	3.03	5.4

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record.		Maximum Observed Precipitation.					Annual Precipitation.			Observed Precipitation, in Percentages of Mean Annual Rainfall.							Main references.
			For day or less.	For month and year.	Last year included.	Number of Minutes.			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.	Mean daily intensity of rainfall.	Maximum During Short Periods.			Mean for month of maximum record.	Maximum.		Annual.			
						5.	15.	60.							Number of Minutes.				In 24 hours.	In other calendar month.	Minimum.	Mean.	Maximum.	
															5.	15.	60.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
241	San Bernardino, Calif.	1 054		49	1919					15.5	5.5	16.1	37.1	2.2				22.3		96.4	84	100	231	2
242	Squirrel Inn, Calif.	5 220		17	1923				16.7	50.3	19.6	39.0	82.2					24.9	62.7	126.6	52	100	210	2
243	Barstow, Calif.	2 105		24	1919					3.9	0.3	4.1	11.0	0.7				13.0		95.0	20	100	269	2
244	Mohave, Calif.	2 751		38	1914					7.3	0.2	4.9	14.6					20.2		149.0	4	100	302	2
245	Tehachapi, Calif.	3 964		39	1914					6.3	0.0	10.3	21.4					24.0		91.0	39	100	207	2
246	Bakersfield, Calif.	404		33	1922					3.2	3.0	5.7	9.3					19.8		56.1	53	100	164	2
247	Death Valley, Calif.	178		16	1927					1.9	0.4	1.8	4.8					17.2		105.8	22	100	250	2
248	Independence, Calif.	3 930		36	1919					14.0	1.4	9.2	21.6	2.7				21.6		230.0	21	100	264	2
249	Lemon Cove, Calif.	900		24	1922					15.2	6.2	15.4	30.6	2.6				19.8		97.3	40	100	199	2
250	Fresno, Calif.	335	40	47	1927					2.9	0.5	3.9	9.7	0.3				20.3	39.9	96.2	40	100	242	1; 2
251	Yosemite, Calif.	3 900		18	1922					24.6	17.3	34.8	69.1					21.3		70.7	50	100	199	2
252	Merced, Calif.	173		51	1922					6.0	4.3	11.2	23.8	2.4				21.5		71.4	38	100	213	2
253	Lick Observatory, Calif.	4 209		42	1922					23.8	15.8	30.4	60.1	1.3				19.5		115.2	52	100	189	2
254	Stockton, Calif.	23		56	1922					11.5	6.4	14.4	29.4					19.0		79.9	44	100	183	2
255	Mokelumne Hill, Calif.	1 550		41	1922					23.0	15.2	31.4	54.9	1.6				19.4		73.4	48	100	176	2
256	Tamarack, Calif.	8 000		23	1922					39.8	27.8	50.6	94.3					22.8		78.7	55	100	167	2
257	Placerville, Calif.	1 920		43	1920					20.6	19.4	34.2	74.4					19.0		88.5	45	100	197	2
258	Sacramento, Calif.	71	25	79	1927	0.29	0.64	0.94	7.2	15.0	7.8	18.3	34.9	1.7	1.6	3.5	5.2	19.6	38.4	82.0	43	100	190	1
259	Folsom, Calif.	270		47	1927					21.6	8.5	20.3	39.2					20.3		96.8	40	100	192	1; 2
260	Auburn, Calif.	1 360		54	1923					23.1	16.6	33.7	60.6					19.7		68.5	49	100	180	2
261	Nevada City, Calif.	2 580		60	1923					41.9	25.7	53.0	102.8					18.6		79.0	49	100	194	2
262	Lake Spaulding, Calif.	4 660		39	1923					49.0	36.9	69.8	119.4					20.0		70.4	53	100	182	2
263	Tahoe, Calif.	6 230		15	1923					21.4	21.5	31.3	40.4					24.0		97.5	62	100	129	2
264	Truckee, Calif.	5 815		30	1920					20.9	6.7	22.2	40.6					22.2		78.6	57	100	167	2
265	Lewer's Ranch, Nev.	6 262		27	1915					21.5	10.7	25.2	48.5					21.0		85.8	43	100	193	2
266	Reno, Nev.	4 524	39	39	1926				2.7	6.8	4.7	8.2	13.7	2.0				20.0	33.0	83.0	57	100	167	1
267	Winnemucca, Nev.	4 346	36	51	1920				1.9	5.2	4.9	8.4	16.4	2.6				21.4	19.0	62.3	58	100	219	2
268	Sewanee, Nor.	6 000		24	1920					6.7	5.2	9.6	16.4	3.8				15.1		54.8	54	100	177	2
269	Clover Valley, Nev.	6 000		27	1920					6.7	7.5	13.8	24.4					15.1		63.0	54	100	177	2
270	Wells, R. E., Nev.	4 675		16	1915					4.6	2.7	9.1	22.4	4.7				14.1		87.2	37	100	246	2
271	Kelton, N. W., Utah.	4 230		43	1920					3.7	1.5	6.4	14.5	3.6				10.0		57.8	23	100	226	2
272	Cortine, Utah.	4 240		51	1920					5.4	3.4	12.6	22.3	2.4				7.8		42.9	43	100	177	2
273	Salt Lake City, Utah.	4 408	54	54	1927				2.7	5.8	3.2	7.1	15.4	1.1				16.1	16.4	79.4	45	100	242	2
274	Strawberry Tunnel, Utah.	7 606		13	1927					6.0	12.6	21.3	28.3	1.1				11.5		28.1	59	100	132	2
275	Modena, S. W., Utah.	5 417		27	1927				2.3	4.0	1.1	3.6	8.2	1.5				12.1	30.4	41.0	27	100	227	1; 4
276	St. George, S. W., Utah.	2 800		28	1927					10.2	3.5	8.7	16.7	2.8				13.3		117.4	40	100	215	2
277	Alton, S., Utah.	9 950		19	1920					12.0	12.2	22.7	34.2	1.9				17.4		52.6	54	100	153	2
278	Mosh, B. E., Utah.	4 000		32	1920					6.6	4.3	9.2	19.0	1.9				9.2		71.8	47	100	174	2
279	Fort Duchesne, Utah.	5 941		33	1920					5.9	4.4	7.1	12.1	2.5				15.9		83.1	62	100	170	2
280	Steamboat Springs, Colo.	6 662		32	1922					6.0	13.9	22.9	32.1	1.1				8.6		61.0	64	100	146	2
281	Grand Junction.	4 562	39	39	1922	0.39	0.59	0.74	2.5	5.0	3.6	7.4	14.9	1.4	4.6	7.0	8.8	10.2	29.8	45.3	64	100	168	2
282	Durango, S. W., Colo.	6 550		27	1922					7.4	8.9	20.0	36.3	1.2				9.0		37.0	44	100	171	2
283	Wagon Wheel Gap, Colo.	9 134		20	1922					6.5	7.3	16.1	25.0	1.1				16.0		46.4	62	100	161	2
284	Gunnison, Colo.	7 670		31	1922					4.2	6.2	9.8	13.4	1.5				14.7		42.9	62	100	137	2
285	Marshall Pass, Colo.	10 670		18	1921					6.6	12.5	17.4	23.6					11.0		38.3	73	100	137	2
286	Leadville, Colo.	10 248		35	1922					7.7	11.7	16.7	23.8	0.9				13.2		46.1	70	100	154	2
287	Long's Peak, Colo.	9 950		19	1920					7.8	13.8	21.4	29.3	1.2				14.3		50.0	64	100	159	2
288	Greeley, Colo.	4 649		35	1922					6.3	5.4	12.6	20.9	1.8				13.6		50.0	43	100	166	2
289	Denver, Colo.	5 292	(25½ (50½	56	1927	0.91	1.54	2.20	6.5	8.6	7.7	14.3	23.0	1.2	6.4	10.3	15.4	23.3	45.4	60.0	54	100	161	1
290	Pike's Peak, Colo.	14 111		18	1894					6.0	13.9	22.9	32.1	1.1				8.6		58.3	64	100	146	2
291	Lake Moraine, Colo.	10 265		28	1922					19.5	18.5	24.7	33.5	1.1				13.1		86.9	75	100	136	2
292	Colorado Springs, Colo.	6 000		30	1922					6.5	7.3	15.0	24.9	1.3				10.4		50.8	52	100	168	2
293	Pueblo, Colo.	4 695	35	54	1927	0.45	0.99	2.07	2.9	11.9	6.1	11.5	20.3	1.5	3.9	8.0	18.0	11.4	25.5	103.4	53	100	177	2; 1
294	Las Animas, Colo.	3 899		58	1922					7.5	3.8	12.3	19.3	3.3				12.5		61.2	31	100	157	2
295	Raton, N. Mex.	6 660		26	1921					11.3	8.0	16.0	23.8	1.9				19.3		70.7	50	100	149	2
296	Anchor Mine, N. Mex.	6 900		10	1920					11.3	13.9	25.5	41.8	0.8				19.7		70.7	59	100	202	2
297	Tate Canyon, N. Mex.	8 929		13	1921					7.1	12.2	21.5	28.1	1.0				15.8		35.8	57	100	122	2
298	Fort Union, N. Mex.	6 567		13	1921					6.0	7.1	14.1	21.6					15.5		48.3	54	100	144	2
299	Santa Fe, N. Mex.	7 013	57	70	1927				2.5	7.9	5.0	14.1	24.8	1.1				15.4	17.7	56.1	35	100	176	1
300	Albuquerque, N. Mex.	4 104		49	1921					6.3	3.6	8.7	16.4	2.1				9.7		103.2	39	100	188	2
301	San Marcial, N. Mex.	4 447		29	1920					13.9	1.1	9.2	24.6	3.8				20.3		129.3	12	100	267	2
302	Roswell, N. Mex.	3 576	27	27	1920	0.19	0.53	1.40	5.6	9.6	4.9	14.1	22.7	1.8	1.3	2.5	5.3	13.6	39.7	86.2	35	100	207	2
303	La Cruces, N. Mex.	3 895		66	1920					5.8	2.5	8.5	17.1	2.4				20.1		68.4	41	100	201	2
304	Fort Bayard, N. Mex.	6 152		51	1920					9.6	5.8	15.5	31.2	1.6				20.6		62.0	37	100	201	2
305	Fort Apache, Ariz.	5 209		45	1921					11.8	8.1	18.3	33.1	1.4				18.6		50.8	48	100	181	2
306	Pinaleño, Ariz.	6 907		42	1920					12.8	15.7	28.3	43.9	0.8				12.4	12.7	46.3	59	100	193	2
307	Grand Canyon, Ariz.	6 806		24	1926					8.5	10.7	18.2	26.7	1.7				8.5		46.8	59	100	202	2; 15
308	Fort Mohave, Ariz.	604		24	1913					4.1	1.3	4.5	21.6	0.7				17.5		219.5	11	100	420	2
309	Prescott, Ariz.	5 320		53	1920					8.8	10.0	17.8	29.0	1.6				16.6		49.5	56	100	222	2
310	Phoenix, Ariz.	1 108	32	49	1920		0.0	0.0		3.3	2.0	4.7	9.6	2.0				16.1	66.7	98.5	30	100	202	1; 2
311	Pinal Ranch, Ariz.	4 520		29	1920					14.9	11.8	25.3	58.4	1.8				14.5		58.6	47	100	231	2
312	Tucson, Ariz.	2 376	27	27	1920	0.19	0.53	1.40	5.6	9.6	4.9	14.1	22.7	1.8	1.3	2.5	5.3	13.6	39.7	86.2	35	100	207	2
313	Yuma, Ariz.	141	57	57	1926				4.0	6.2	0.6	3.4	11.4	0.9				17.0	117.5	182.3	18	100	335	1
314	Chihuahua, Mexico.	4 670		13	1918					7.6	9.4	15.0	20.6					17.4		51.0	63	100	136	4
315	Mazatlan, Mexico.			26	1920					23.6	15.4	25.1	37.9					20.4		68.0	67	100	202	4
316	Monterey, Mexico.	1 722		26	1924					17.2	3.4	19.5	37.9					22.6		86.0	17	100	194	4
317	Leon, Mexico.	5 930		40	1920					16.1	12.4	25.2	36.7					18.0		64.0	49	100	146	4
318	Mexico City, Mexico.	7 530		41	1920					10.1	8.4	15.1	24.0					20.3		59.0	54	100	164	4
319	Puebla, Mexico.	7 050		41	1920					14.8	22.2	33.5	50.1					20.3		42.6	67	100	150	4
320	Oaxaca, Mexico.	5 130		40	1920					14.8	22.2	33.5	50.1					20.3		44.0	67	100	150	4
321	Merida, E. E., Mexico.	72		26	1924					17.7	16.0	35.1	64.2					15.5		50.1	46	100	183	4
322	Corozal, Belize.			16	1929					17.5	33.7	52.5	66.5					12.7		33.3	64	100	157	15
323	Belize, Belize.			24	1915					30.2	13.0	30.2	130.5					15.5		50.1	46	100	183	15
324	Setal, Guatemala.	2 430		17	1913					49.9	139.0	193.1	263.6					14.7		25.9	72	100	136	15
325	Chimar, Guatemala.	4 200		20	1913					27.1	68.9	96.0	116.9					14.1		28.3	72	100	122	15
326	Magdalena, Guatemala.	4 229		11						11.1		15.4	19.6					15.5		43.8	72	100	151	15
327	Guatemala, Guatemala.	4 855		29	1913					22.6	25.0	51.8	71.7	0.3				15.5		43.8	48	100	139	15
328	San Francisco, Guatemala	2 430		25						49.0		108.2	238.8					10.7		29.0		100	141	15
329	San Salvador, Salvador.	(2 155 (Maxima		23	1920					18.0	58.1	68.3	83.4					16.6		26.3	85	100	122	15
330	Rivas, Nicaragua.	150		21	1900					33.8	31.8	69.8	123.4	0.8				24.9		48.5	46	100	177	15
331	Greytown, S. E., Nicaragua.	50±		6	1900					55.4	201.6	254.9	290.9					14.3		21.8	79	100	117	15
332	Sarapiqui, Costa Rica.	2 750		6	1903					52.2	180.2	209.4	268.5					14.3		25.0	86	100	126	15
333	San Jose, Costa Rica.	3 720		34	1904					28.7	19.5	70.8	154.3	0.6				16.6		40.5	29	100	218	4
334	Balboa, Canal Zone.	1116 (Maxima	16	24	1921				7.2	20.3	45.6	69.4	91.4					14.8	10.4	29.2	66	100	132	15
335	Gamboa, Canal Zone.		16	16	1921				6.6	28.4	62.0	80.8	136.2					13.0	7.2	31.3	68	100	159	15
336	Colon, Canal Zone.		36	36	1927				8.5	42.6	92.5	127.3	183.4					15.3	6.7	35.4	73	100	224	4; 15
337	Porto Bello, Panama.	50±	10	10	1921					10.9	58.2	136.6	160.0	237.3				10.5	6.8	36.5	86	100	148	15
338	Cartagena, Colombia.	50±		12								25.6												15
339	Maracaibo, Venezuela.	20	9	11						4.4		20.9						15.0	21.1			100		15
340	Caracas, Venezuela.	3 420	16	35	1925					3.3	8.4	22.6	31.7	47.4				13.5	10.4	26.5	71	100	150	4; 15
341	Ciudad Bolivar, Venezuela.	125	7	9						4.1			47.1					8.6	10.2			100		15
342	Calabozo, Venezuela.	328	5	7						4.1			47.1					2.5	8.7			100		15
343	Merida, Venezuela.	5 384	9	11						3.9			71.2					14.1	5.4			100		15
344	Bogota, Colombia.	8 700		50	1923					15.5	25.4	41.6	56.8					9.8		37.2	61	100	136	4
345	Medellin, Colombia.	4 950		7								58.6										100		15
346	Buenaventura, Colombia.	50±		7								280.6										100		15
347	Quito, Ecuador.	9 350		14	1915					10.3	38.0	43.1	54.0					16.8		23.9	88	100	125	15
348	Guayaquil, Ecuador.	50±		14	1899					16.1	28.0	41.7	46.0					25.7		41.7	75	100	119	15
349	Lima, Peru.	550	9	18					0.2			1.9						21.6	10.5			100		15
350	Chuco, Peru.	11 500		12	1921					12.7	13.1	31.7	45.8					17.8		40.0	41	100	144	15
351	Callloma, Peru.	13 000		17								26.1										100		15
352	Arequipa, Peru.	8 641		17	1926					8.1	0.02	12.3	12.6					40.5		188.2	5	100	279	4
353	La Joya, Peru.	4 140		9								0.2										100		15
354	Mollendo, Peru.	50±		10								0.4										100		15
355	La Paz, Bolivia.	11 910		5								21.2										100		15
356	Oruro, Bolivia.	12 122		4								2.1										100		15
357	Sucre, Bolivia.	9 344		32								20.2										100		15
358	La Serena, Chile.	115		12	1920					9.3	0.3	5.4	16.2					16.1		173.0	6	100	300	4
359	Santiago, Chile.	1 702		55	1921					13.9	5.0	14.2	32.3					22.3		98.0	35	100	223	4
360	Punta Gaiero, Chile.	131		23	1921					27.2	20.9	37.5	57.5					14.2		31.1	61	100	157	4
361	Valdivia, Chile.	49		52	1921					44.7	11.7	106.3	140.7					15.4		32.0	67	100	132	4
362	Evangelistes Is., Chile.	180		23	1921					19.7	85.5	121.0	137.0					9.2		16.3	71	100	113	4
363	Punta Arenas, Chile.	95		34	1921					6.4	8.7	15.3	22.2					9.2		28.9	67	100	145	4
364	South Orkney Is.	23		18	1923					3.4	11.0	15.9	20.7					10.8		23.4	69	100	130	4
365	South Georgia Is.	13		21	1926					13.9	40.4	51.3	66.5					9.8		27.1	79	100	130	4
366	Stanley, Falkland Is.	6		17	1920					6.5	20.7	27.0	36.8					10.2		24.1	77	100	136	4

(Continued.)

Station No.	JANUARY.			FEBRUARY.			MARCH.			APRIL.			MAY.			JUNE.			JULY.			AUGUST.			SEPTEMBER.			OCTOBER.			NOVEMBER.			DECEMBER.		
	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.	Mean month.	Maximum. 24 hours.	Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
241	3.57	15.5	3	3.02	12.2	2	2.80	9.9	1	1.20	5.7	1	0.57	3.2	0.10	1.0	0.04	0.3	0.16	2.2	0.22	2.4	0.62	2.7	1.30	7.5	2.4	10.8								
242	5.77	50.3	6	4.64	16.3	17	3.57	20.6	5	0.56	6.3	2	2.02	6.5	0.14	1.1	0.04	0.5	0.19	1.0	0.70	4.0	1.52	6.5	2.72	9.2	5.08	18.5								
243	0.79	3.4	0	0.77	2.5	0.62	0.5	0.5	0.5	0.23	0.6	0.6	0.06	0.4	0.14	3.0	0.25	1.3	0.14	0.8	0.22	1.5	0.40	3.4	0.25	0.6	0.63	2.9								
244	0.92	3.3	0.94	0.7	0.8	0.7	0.9	0.2	0.1	0.21	1.0	0.04	0.5	0.02	1.0	0.08	1.7	0.08	0.9	0.21	2.2	0.34	2.2	0.98	7.2											
245	1.52	4.9	2.66	6.3	1.82	5.1	1.18	2.7	0.39	1.7	0.03	1.0	0.08	1.7	0.14	1.2	0.41	1.7	0.14	1.2	0.40	3.7	0.64	4.2	0.98	7.2										
246	1.13	3.2	0.80	2.4	1.06	2.3	0.41	3.0	0.39	2.4	0.05	0.4	0.02	0.4	0.01	0.2	0.15	1.1	0.35	2.0	0.52	2.5	0.76	1.9												
247	0.49	1.5	0.31	1.9	0.19	1.1	0.02	0.2	0.07	0.4	0.05	0.6	0.02	0.6	0.04	0.4	0.14	0.8	0.09	0.5	0.27	1.7	0.09	0.6												
248	0.77	2.6	0.61	1.6	0.38	1.0	0.11	0.8	0.24	1.0	0.05	0.6	0.02	0.6	0.04	0.4	0.15	0.9	0.28	2.1	0.18	1.3	0.31	1.6												
249	2.87	7.6	3.05	15.2	3.01	9.8	1.25	3.1	0.55	3.2	0.11	1.0	0.02	0.2	0.01	0.2	0.25	2.4	0.62	3.8	1.27	5.8	1.96	6.1												
250	1.75	1.7	5.2	1.47	2.8	4.6	1.04	3.5	4.2	0.90	1.7	3.0	0.43	3.2	2.9	0.10	1.7	1.5	0.01	0.3	0.2	0.22	1.1	1.8	0.69	2.4	3.2	1.00	2.9	9.5	1.44	2.2	4.5			
251	7.40	24.6	5	5.87	13.9	6.19	21.0	1.88	4.9	1.47	7.5	0.55	3.1	0.19	1.4	0.29	1.4	1.17	7.5	1.55	3.7	10.7	2.36	10.1	6.37	13.1										
252	2.41	8.0	1	1.66	4.4	1.94	6.6	0.89	5.6	0.54	2.5	0.11	1.7	0.01	0.2	0.02	0.5	0.22	1.8	0.49	3.0	1.15	5.8	1.76	6.6											
253	1.84	10.3	1	0.73	12.7	0.55	4.2	0.59	3.1	1.47	6.3	0.11	1.7	0.01	0.2	0.02	0.3	0.47	5.2	1.41	6.4	2.70	10.3	5.91	23.6											
254	3.06	13.9	1	2.32	7.1	2.19	7.1	0.99	6.3	0.64	4.8	0.18	2.9	0.04	0.5	0.08	0.3	0.58	3.7	0.54	3.6	1.47	6.4	2.74	11.5											
255	5.79	23.0	5	5.05	13.3	5.77	15.7	2.45	10.1	1.42	4.0	0.26	2.9	Tr.	0.03	0.02	0.2	0.65	4.1	1.67	5.4	3.31	15.2	5.10	16.8												
256	11.50	29.8	7	7.41	15.9	7.73	31.6	2.82	7.0	2.70	8.3	1.29	7.4	0.83	4.2	0.53	3.2	1.10	6.1	2.43	6.3	4.81	14.4	7.42	9.8											
257	8.44	28.3	6	6.62	12.5	7.32	31.5	3.65	17.5	2.08	8.0	0.56	2.5	0.22	0.7	0.42	2.5	0.75	4.2	2.14	5.4	3.41	8.5	17.6	6.06	22.6										
258	3.59	3.2	15.0	2.91	2.6	8.5	2.67	2.9	10.0	1.48	7.2	14.2	0.75	1.9	2.2	0.13	0.8	1.4	0.02	0.1	0.01	0.2	0.26	2.1	3.6	0.82	2.1	6.0	1.97	8.2	11.3	3.73	2.6	13.4		
259	5.06	23.1	3	3.97	10.1	3.75	11.1	1.99	11.4	1.06	4.6	0.31	1.6	0.01	0.2	0.02	0.6	0.46	3.6	1.19	5.7	2.45	10.6	4.23	13.1											
260	6.62	23.1	5	5.62	14.6	5.31	16.7	2.77	13.0	1.36	4.7	0.38	2.4	0.04	0.2	0.03	0.3	0.58	4.0	1.72	5.9	3.51	15.2	5.85	16.6											
261	10.57	35.1	8	8.17	19.2	8.26	24.6	4.22	22.5	2.31	10.8	0.62	3.3	0.04	0.7	0.05	0.7	0.33	6.0	2.42	13.0	5.67	21.5	9.83	41.9											
262	13.85	49.0	11	13.34	34.7	11.76	35.5	8.84	18.3	3.70	14.8	1.14	4.6	0.11	0.7	0.17	0.9	1.48	5.3	2.87	14.3	7.04	20.4	10.07	22.4											
263	7.52	21.2	5	5.99	14.2	3.39	8.6	1.78	4.4	1.16	3.4	0.53	1.8	0.32	1.4	0.16	1.0	0.73	4.7	1.79	4.2	2.74	7.6	6.65	10.9											
264	5.81	20.9	4	4.95	16.4	4.31	20.5	1.77	12.7	1.07	4.6	0.31	1.3	0.19	2.2	0.19	1.1	0.34	2.8	0.92	6.5	2.10	8.9	4.30	16.2											
265	5.27	21.5	3.54	19.1	3.65	16.8	1.44	4.2	1.56	3.7	0.44	2.7	0.22	1.4	0.34	1.8	0.64	2.2	1.23	4.4	2.99	9.0	3.84	10.6												
266	4.64	2.4	9.2	4.15	5.0	0.89	2.9	4.1	2.75	5.2	0.47	1.9	0.30	1.2	0.27	1.4	0.32	3.2	0.77	2.4	2.17	6.9	1.09	2.1	2.61	7.1										
267	1.14	1.6	3.1	0.88	0.7	2.0	0.96	1.0	5.2	0.80	0.9	2.3	0.68	2.6	0.61	1.6	2.4	0.18	0.9	0.17	0.6	1.8	0.37	1.0	1.5	0.65	0.7	1.9	0.76	1.1	2.8	0.97	1.1	3.4		
268	2.01	6.2	3.1	1.59	2.1	3.5	1.82	2.0	1.40	1.07	3.3	1.17	3.3	0.37	1.2	0.22	1.1	0.54	2.1	0.57	2.1	0.90	2.3	1.16	3.5	0.87	1.2	2.8	0.90	1.2	2.9	0.80	1.2	2.9		
269	2.09	5.7	2.02	8.1	1.43	3.3	1.13	2.3	1.25	4.1	0.71	3.7	0.44	1.9	0.53	1.4	0.56	1.6	0.78	2.8	1.56	6.9	1.36	4.5												
270	1.33	3.7	2.37	6.2	1.33	3.3	1.43	3.7	0.64	2.3	0.10	1.0	0.02	0.2	0.03	0.2	0.23	1.0	0.23	1.4	0.56	2.4	0.44	1.2	4.3	4.3										
271	0.69	2.5	0.64	3.7	0.51	2.2	0.61	2.7	0.78	3.0	0.45	2.3	0.42	2.1	0.22	1.5	0.46	2.2	0.55	3.0	0.42	1.9	0.69	3.3												
272	1.34	4.0	1.27	3.6	1.32	4.1	1.11	3.5	1.46	5.8	0.65	3.6	0.48	2.5	0.58	2.8	0.76	3.2	1.17	4.1	0.98	5.4	1.40	5.5												
273	1.41	4.1	1.37	4.3	2.04	3.8	1.02	1.4	4.4	0.62	2.9	0.55	3.1	0.64	2.4	0.84	2.2	0.56	1.8	0.50	1.5	2.84	5.8	1.42	4.4											
274	2.45	6.9	2.28	4.0	2.34	5.2	1.54	2.8	1.37	2.9	0.92	2.5	1.55	2.9	1.68	3.9	1.82	3.2	2.14	4.8	1.51	6.8	1.64	4.6												
275	1.98	3.5	1.89	3.7	2.37	5.1	1.54	2.6	1.08	4.6	0.61	3.1	1.02	2.2	1.54	2.1	0.97	2.3	1.17	3.2	1.06	4.2	1.60	4.2	0.56	2.0	0.66	1.6	2.1							
276	0.88	3.0	1.04	2.7	0.71	2.0	0.49	2.2	0.16	1.7	0.10	0.9	0.02	0.2	0.04	0.2	0.05	0.3	0.07	0.4	0.03	0.2	0.04	0.2	0.03	0.2	0.04	0.2	0.03	0.2	0.04	0.2	0.03	0.2	0.04	0.2
277	3.04	10.8	2.58	7.6	3.95	12.0	1.19	3.6	1.01	2.8	0.48	2.2	0.95	4.1	0.95	2.1	0.95	6.1	1.62	7.8	1.00	5.9	1.55	4.9												
278	0.91	3.5	0.98	2.6	0.77	2.1	0.61	2.4	0.74	2.6	0.31	1.5	0.95	2.1	0.60	2.0	0.61	4.0	0.95	4.0	0.97	1.6	0.93	2.7												
279	0.54	2.4	0.42	1.6	0.45	3.1	0.60	2.0	0.72	2.0	0.34	2.5	1.5	0.65	2.5	1.3	0.58	3.0	0.69	3.0	0.37	1.8	0.60	2.0												
280	2.51	5.5	2.67	4.2	1.89	3.7	2.06	5.1	1.91	3.9	1.34	5.0	1.40	5.0	1.59	5.4	1.53	3.0	1.78	6.0	1.58	2.6	2.55	5.6												
281	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.7	1.6	1.7	1.6	1.7	1.6	1.7	1.6	1.7	1.6	1.7	1.6
282	1.84	5.9	1.85	7.0	1.94	4.8	1.48	3.9	1.07	3.2	0.80	2.1	2.01	5.2	2.12	4.8	1.77	4.2	1.99	7.0	1.77	3.4	1.81	7.4												
283	1.82	5.9	1.87	3.0	1.52	4.0	1.49	4.2	1.15	3.1	0.77	2.0	1.87	4.8	1.97	4.4	1.64	3.9	1.84	6.5	1.64	3.2	1.68	6.9												
284	0.80	2.6	0.76	2.1	0.60	2.1	0.55	3.1	0.78	2.7	0.64	2.2	1.44	4.2	1.32	2.8	0.81	2.7	0.61	1.9	0.56	3.6	0.71	1.9												
285	1.74	4.1	1.69	3.3	1.82	5.0	1.40	3.7	1.07	3.3	1.17	3.3	1.60	3.9	1.83	3.8	0.94	2.2	1.15	4.0	1.02	3.2	1.19	4.6												
286	1.21	3.3	1.51	4.7	1.61	4.6	1.74	4.9	1.19	2.6	0.97	3.0	2.20	7.7	1.90	6.0	1.17	4.1	1.11	2.5	0.84	2.3	1.22	3.2												
287	1.98	5.3	2.20	12.6	2.20	7.2	1.05	3.2	0.97	3.4	1.55	3.4	1.55	3.9	1.95	4.5	1.55	3.7	1.55	5.2	1.55	3.7	1.55	6.9												
288	0.32	2.3	0.41	1.6	0.73	3.1	1.71	6.3	2.47	5.8	1.41	3.3	1.85	5.1	1.13	3.9	0.96	4.1	0.92	3.1	0.33	1.0	0.61	4.9												
289	0.43	1.2	2.3	0.51	0.9	1.3	1.02	2.8	8.2	3.33	6.5	8.6	1.40	2.2	5.0	1.75	2.8	5.2	1.44	3.0	3.9	1.02	2.7	3.8	1.07	3.1	1.9	0.73	2.3	5.2						
290	1.58	4.3	1.49	3.9	2.06	4.3	2.57	13.1	3.81	12.3	1.40	2.2	5.0	1.75	2.8	5.2	1.44	3.0	3.9	1.02	2.7	3.8	1.07	3.1	1.9	0.73	2.3	5.2								
291	0.74	2.4	0.69	2.0	0.96	2.4	0.89	2.1	0.74	2.0	2.61	7.5	4.44	8.1	3.66	11.0	1.66	3.9	1.44	2.8	0.53	2.5	0.89	2.0												
292	0.23	0.9	0.39	1.8	0.67	3.0	1.74	7.6	2.25	7.3	1.93	7.5	2.01	5.2	2.12	5.0	1.05	4.6	0.60	2.1	0.34	2.0	0.31	2.6												
293	2.07	5.9	2.29	5.2	0.64	2.0	1.91	5.2	1.54	2.5	3.81	7.2	2.04	2.1	2.04	2.1	2.04	2.7	1.53	3.0	1.53	3.0	1.53	3.0												
294	0.19	0.8	0.45	2.1	0.53	2.4	1.54	7.5	1.92	5.4	1.42	4.1	2.17	6.3	1.02	0.0	1.00	4.6	0.69	3.7	0.32	2.0	0.44	3.7												
295	0.24	0.8	0.71	2.1	0.66	4.4	1.59	6.0	1.89	4.3	2.12	9.4	3.09	11.3	2.12	5.1	1.67	7.4	1.02	3.6	0.45	2.0	0.48	2.7												
296	0.91	3.1	1.01	2.6	0.89	2.9	0.94	4.2	1.73	3.0	0.97	3.5	2.08	7.4	2.08	4.2	1.67	4.3	1.04	3.1	0.72	2.3	0.97	4.3												
297	1.36	5.1	1.51	2.6	1.85	3.9	2.27	4.1	1.76	3.0	1.67	4.9	3.41	7.7	2.67	5.3	1.52	3.4	1.04	3.1	0.72	1.7	1.15	2.0												
298	0.43	2.0	0.49	1.6	0.57	2.0	0.49	1.6	0.49	1.6	0.57	2.0	0.57	2.0	0.57	1.6	0.57	2.0	0.57	2.0	0.57	2.0	0.57	2.0												
299	0.67	1.2	3.0	0.75	0.8	5.2	0.84	1.2	2.6	1.09	1.4	4.8	1.31	2.2	4.3	1.06	2.0	4.2	2.37	2.1	7.4	2.17	1.0	7.9	1.46	2.5	5.4	1.17	1.8	6.6	0.71	1.3	3.5	0.73	0.9	2.3
300	0.57	1.3	2.7	0.62	1.1	1.6	0.67	1.2	4.4	1.55	2.4	6.1	1.10	2.6	4.1	1.03	2.0	4.1	2.27	2.2	7.9	2.17	1.0	7.9	1.46	2.5	5.4	1.17	1.8	6.6	0.71	1.3	3.5	0.73	0.9	2.3
301	0.32	2.0	0.36	2.4	0.31	1.6	0.33	1.6	0.29	2.0	0.59	2.4	1.89	8.3	1.86	11.9	1.55	8.0	0.82	3.1	0.53	4.2	0.43	2.5												
302	0.42	1.2	1.6	0.52	1.2	2.6	0.69	2.3	5.2	0.99	2.6	4.6	2.9	6.2	2.45	6.5	1.95	5.6	1.95	6.2	1.95	6.1	0.65	1.2	1.8	1.8										
303	0.32	2.0	0.36	4.5	0.31	2.0	0.19	1.0	0.24	1.7	0.57	3.6	1.72	5.2	1.71	5.8	1.30	3.2	0.70	2.8	0.30	1.2	0.42	2.4												
304	0.96	3.3	1.00	5.7	0.79	4.3	0.41	2.0	0.31	1.5	0.72	3.7	3.20	9.6	3.27	8.7	1.94	5.9	1.18	8.2	0.77	3.8	0.92	4.6												
305	1.56	5.0	1.62	5.2	1.53	6.8	0.81	5.0	0.54	2.7	0.49	3.3	3.04	9.2	2.40	8.3	1.54	3.9	1.20	4.7	1.12	5.7	1.43	5.5												
306	2.82	12.5	2.75	8.4	2.56	6.7	1.23	4.5	1.10	7.5	0.41	1.9	3.76	9.2	3.76	9.2	1.43	3.7	1.00	4.6	0.84	4.2	1.76	7.5												
307	1.48	4.4	1.61	4.2	1.50	3.7	0.95	4.0	0.70	2.0	0.26	1.0	2.71	6.0	2.48	7.0	1.52	4.8	1.24	8.5	1.42	7.0	1.87	7.5												
308	0.43	2.0	0.49	1.6	0.57	2.0	0.49	1.6	0.49	1.6	0.57	2.0	0.57	2.0	0.57	1.6	0.57	2.0	0.57	2.0	0.57	2.0	0.57	2.0												
309	1.78	7.8	1.82	7.9	1.81	8.0	0.86	3.8	0.51	1.7	0.28	2.1	2.95	8.5	3.05	6.7	1.10	4.9	0.98	4.2	1.12	4.7	1.50	7.4												
310	2.58	7.3	2.71	11.4	2.67	13.2	1.44	5.4	1.10	6.1	0.27	2.5	2.77	7.2	2.69	7.2	2.09	6.7	1.69	6.7	1.77	6.1	1.77	6.1												
311	3.68	14.9	2.83	13.4	2.44	10.0	0.87	5.5	0.43	3.4	0.45	2.4	2.83	6.3	3.54	7.5	2.08	5.5	1.77	4.5	2.10	9.7	2.64	10.9												
312	1.42	5.1	1.52	4.2	1.42	3.2	1.14	3.2	0.86	2.5	0.34	2.0	2.45	6.2	2.45	6.2	1.95	5.2	1.56	5.2	1.56	6.1	0.65	1.2												
313	4.23	2.2	5.8	0.42	2.5	3.4	3.3	0.09	0.8	0.9	0.049	0.9	0.092	0.6	0.6	0.201	3.1	1.4	0.58	4.6	2.2	0.34	3.6	3.0	0.22	1.7	1.7	0.26	1.7	2.4	0.01	2.4	4.4			
314	0.31	0.8	0.38	2.5	0.31	2.9	0.14	0.7	0.19	0.5	2.05	4.8	3.83	7.7	3.45	6.2	2.62	7.6	0.85	2.2	0.69	3.0	0.18	0.9												
315	0.75	4.1	0.46	3.2	0.15	2.9	0.00	1.9	0.09	1.8	1.80	10.0	2.15	8.2	2.01	8.6	1.42	17.2	1.42	16.0	0.37	2.2	0.70	5.5												
316	0.57	2.4	0.56	5.0	0.74	2.9	1.11	4.3	1.18	3.2	0.57	2.5	2.25	6.4	2.25	6.4	1.42	5.2	1.42	5.2	1.42	5.2	1.42	5.2												
317	0.39	2.6	0.30	1.3	0.43	1.9	0.24	1.7	1.25	6.4	4.54	16.1	0.21	1.2	5.24	9.8	4.55	13.1	1.43	4.8	0.47	1.6	0.40	2.1												
318	0.33	2.0	0.31	2.7	0.40	1.9	1.03	3.8	1.48	4.2	0.57	2.5	2.25	6.4	2.25	6.4	1.42	5.2	1.42	5.2	1.42	5.2	1.42	5.2												
319	0.38	1.6	0.31	2.1	0.47	1.9	1.03	3.8	1.48	4.2	0.57	2.5	2.25	6.4	2.25	6.4	1.42	5.2	1.42	5.2	1.42	5.2	1.42	5.2												
320	0.91	3.0	0.71	3.1	1.05	3.1	0.90	5.4	2.76	8.0	6.78	12.2	4.43	9.3	5.77	13.0	5.47	17.7	3.22	7.8	1.83	6.1	1.27	6.2												
321	2.57	6.5	1.47	4.1	1.24	4.1	0.36	1.6	0.99	10.9	8.74	15.0	6.95	15.6	5.80	10.2	6.41	13.2	6.76	17.5	5.19	11.0	2.05	5.1												
322	3.74	19.3	3.25	9.2	3.45	7.8	2.26	4.7	1.26	4.2	0.39	2.0	3.95	10.3	3.95	10.3	2.34	6.2	2.00	7.2	2.00	7.2	2.00	7.2												
323	15.24	28.5	11.30	20.3	9.86	29.2	8.54	18.7	13.46	22.1	20.98	36.9	18.11	24.2	15.43	22.0	19.84	30.0	28.46	49.9	18.54	32.9	15.28	32.8												
324	6.60	10.5	4.22	8.2	4.84	10.2	4.28	9.1	8.28	14.6	11.85	18.8	10.11	15.0	7.96	12.0	9.88	15.4	12.54	27.1	8.63	19.3	6.66	9.7												
325	0.26	1.7	0.12	1.2	0.17	0.7	0.4	3.1	2.16	5.4	5.45	15.5	7.98	22.6	7.77	12.2	9.11	14.1	6.76	17.9	5.84	12.0	3.22	5.2												
326	1.80	5.1	1.56	3.3	1.74	3.3	1.24	6.9	6.00	16.8	10.84	15.5	7.98	22.6	7.77	12.2	9.11	14.1	6.76	17.9	5.84	12.0	3.22	5.2												
327	1.50	9.20	19.80	27.0	14.00	19.52	22.35	28.15	49.0	25.04	2.09																									
328																																				
329	0.17	1.2	0.21	1.4	0.59	3.1	1.96	11.1	6.69	12.9	11.13	14.0	12.11	15.2	11.72	14.2	11.03	15.8	10.37	18.0	1.79	4.7	0.59	4.6												
330	0.46	2.1	0.20	1.7	0.26	2.0	0.26	2.0	5.19	21.3	11.83	24.6	7.20	15.0	8.07	20.8	10.17	23.8	4.58	10.9	1.27	3.1														
331	22.50	28.6	10.77	25.2	9.15	10.2	11.20	18.4	17.86	50.9	23.19	46.8	34.41	52.5	27.28	39.0	17.40	36.9	19.98	27.9	36.45	55.4	27.76	41.6												
332	19.56	42.4	9.29	16.6	6.87	10.9	10.67	17.8	19.41	29.2	21.10	25.3	19.

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record. For day or rain.	Years of Record. For month and year.	Years of Record. Last year included.	Maximum Observed Precipitation. Number of Minutes. 5.	15.	60.	Maximum Observed Precipitation. Day of 24 hours.	Maximum Observed Precipitation. Month.	Annual Precipitation. Minimum.	Annual Precipitation. Mean.	Annual Precipitation. Maximum.	Mean daily interval between rainfall.	Observed Precipitation, in Percentages of Mean Annual Rainfall. Maximum During Short Periods. Number of Minutes. A.	15.	60.	Mean for month of maximum record.	Observed Precipitation, in Percentages of Mean Annual Rainfall. Maximum. In 24 hours	Annual. In one calendar month.	Annual. Minimum.	Annual. Mean.	Annual. Maximum.	Main references.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
367	Bahia Blanca, Argentine	82	...	65	1924	...	...	...	12.4	8.1	21.0	50.0	...	...	...	...	...	10.7	...	59.0	39	100	241	4
368	Ajo-General Lavalle, Argentine	49	...	67	1924	...	...	...	14.9	10.0	34.7	71.1	...	...	...	...	...	9.5	...	42.8	53	100	205	4
369	Buenos Aires, Argentine	52	...	94	1924	...	...	...	23.1	19.2	37.1	79.6	...	...	...	...	...	11.7	...	52.8	53	100	214	4
370	Cordoba, Argentine	1 389	...	52	1924	...	...	...	12.4	17.7	27.7	60.0	...	...	...	...	...	16.8	...	44.7	64	100	144	4
371	San Juan, W., Argentine	2 169	...	42	1924	...	...	...	3.0	0.2	3.8	9.7	...	...	...	...	...	20.0	...	97.3	5	100	181	4
372	Tucuman, N., Argentine	1 470	...	45	1924	...	...	...	15.4	16.5	41.9	57.0	...	...	...	...	...	13.9	...	36.7	39	100	136	4
373	Asuncion, Paraguay	305	...	29	1923	...	...	...	18.6	22.6	51.8	78.5	...	...	...	...	...	11.9	...	35.9	44	100	177	4
374	Montevideo, Uruguay	90	...	29	1924	...	...	...	15.6	22.0	38.8	94.5	...	...	...	...	...	9.3	...	49.1	59	100	244	4
375	Curitiba, Brazil	2 979	...	36	1924	...	...	...	16.2	35.5	55.0	75.1	...	...	...	...	...	10.2	...	29.5	65	100	136	4
376	Alto de Serra, Brazil	2 624	...	55	1924	...	...	...	50.1	89.3	141.6	219.0	...	...	...	...	...	8.9	...	35.5	63	100	155	4
377	Rio de Janeiro, Brazil	201	...	73	1925	...	...	...	19.4	28.8	43.2	69.0	...	...	...	...	...	12.3	...	42.5	67	100	150	4
378	Pernambuco, Brazil	97	...	48	1922	...	...	...	58.7	24.9	65.0	178.5	...	...	...	...	...	16.1	...	87.0	38	100	275	4
379	Ceara, Brazil	66	...	72	1920	...	...	...	27.2	16.4	50.3	109.4	...	...	...	...	...	23.9	...	68.4	33	100	194	4
380	Para, Brazil	39	...	27	1924	...	...	...	30.9	64.1	87.9	116.8	...	...	...	...	...	16.1	...	35.4	74	100	134	15
381	Cayenne, Guiana	20	...	41	1907–1	...	...	...	...	...	118.5	...	...	...	...	...	...	...	...	...	...	100	...	15
382	Georgetown, Guiana	6	...	30	1923	...	...	...	26.6	52.7	88.7	121.9	...	...	...	...	...	12.6	...	39.0	59	100	138	4
383	Fort of Spain, Trinidad	69	18	39	1920	...	...	...	3.2	17.5	39.5	63.7	92.6	...	...	...	...	13.9	5.0	27.5	62	100	145	4; 6
384	Saugre Grande, Trinidad	...	...	27	...	...	...	...	...	...	...	111.9	...	...	...	...	...	...	...	...	...	100	...	6
385	St. George, Grenada	512	13	30	1920	...	...	...	4.3	20.9	56.7	76.1	106.0	...	...	...	...	11.0	5.7	27.5	74	100	139	4; 6
386	Barbados Island	182	7	65	1920	...	...	...	4.6	28.4	27.0	54.8	89.3	...	...	...	...	13.9	8.4	51.8	49	100	163	4; 6
387	Fort de France, Martinique	13	...	58	1923	...	...	...	...	...	...	78.8	...	...	...	...	...	...	...	...	...	100	...	6
388	Morne des Cadets, Martinique	1 676	...	9	1923	...	...	...	...	...	...	120.2	...	...	...	...	...	...	...	...	...	100	...	6
389	Christiansted, Virgin Is.	23	...	47	1925	...	...	...	18.7	25.0	46.5	74.0	...	...	...	...	...	9.1	...	40.3	54	100	159	4
390	St. Thomas, Virgin Is.	27	7	17	1924	...	...	...	5.2	...	40.9	...	...	...	...	...	...	15.0	13.0	...	...	100	...	6
391	San Juan, Porto Rico	82	25	27	1925	...	...	...	40.5	15.4	45.9	99.5	79.0	...	...	...	...	9.5	17.3	25.5	76	100	130	4; 6
392	Tres Negre Dam, Porto Rico	2 275	13	13	1924	...	...	...	8.0	...	...	92.0	...	...	...	...	...	16.4	8.6	...	...	100	...	6
393	Santo Domingo, Haiti	60	10	14	1924	...	...	...	7.6	...	...	55.5	...	...	...	...	...	13.1	13.3	...	...	100	...	6
394	Port Au Prince, Haiti	123	41	49	1925	...	...	...	6.1	19.3	32.2	54.0	76.3	0.8	...	...	...	17.3	11.3	35.7	60	100	142	4; 6
395	Morant Point, Jamaica	8	29	50	1919	...	...	...	12.0	...	...	70.2	...	0.6	...	...	...	11.0	17.1	...	...	100	...	6
396	Kingston, Jamaica	24	39	53	1924	...	...	...	8.5	...	...	52.5	...	1.3	...	...	...	8.6	27.5	...	...	100	...	6
397	Hill Gardens, Jamaica	4 900	29	49	1919	...	...	...	28.1	...	...	105.5	...	0.6	...	...	...	9.9	27.2	...	...	100	...	6
398	Mooretown, Jamaica	...	15	19	1919	...	...	...	39.4	...	...	220.8	320.8	0.6	...	...	...	8.6	...	31.4	...	100	144	6
399	Santiago de Cuba	118	16	21	...	...	...	...	14.2	...	...	43.8	...	1.1	...	...	...	15.2	32.7	...	...	100	...	6
400	Isle of Pines, near Cuba	200	10	13	...	...	...	...	6.8	...	...	70.7	...	...	...	...	...	15.6	9.7	...	...	100	...	6
401	Havana, Cuba	161	21	07	1925	...	...	...	10.3	22.6	27.9	52.2	65.2	1.0	...	...	...	4.6	21.4	47.0	...	100	126	4; 6
402	Nassau, Bahama Is.	99	52	56	1920	...	...	...	8.0	23.4	25.5	51.0	79.2	0.8	...	...	...	12.5	15.8	46.0	50	100	156	4; 6
403	Prospect, Bermuda	151	47	55	1924	...	...	...	10.7	21.3	39.6	57.8	87.6	0.6	...	...	...	9.4	18.5	37.0	69	100	152	6; 4
404	Sable Island, Canada	25	...	30	1920	...	...	...	...	17.9	43.4	51.8	67.6	...	...	...	...	7.1	...	34.6	84	100	131	4
405	Charlottetown, Canada	45	...	50	1925	...	...	...	...	19.4	18.5	39.0	56.4	...	...	...	...	10.9	...	26.6	47	100	144	4
406	St. Johns' Newfoundland	125	...	70	1919	...	...	...	...	14.0	42.3	54.4	75.3	...	...	...	...	9.6	...	34.8	59	100	127	4
407	Father Point, Canada	20	...	49	1925	...	...	...	...	8.1	19.1	33.6	47.3	...	...	...	...	13.8	...	24.2	57	100	141	4
408	Montreal, Canada	187	...	52	1925	...	...	...	...	6.6	29.2	40.4	53.6	...	...	...	...	8.6	...	29.2	73	100	131	4
409	Toronto, Canada	379	...	52	1925	...	...	...	...	9.1	24.3	31.2	48.5	...	...	...	...	8.8	...	29.2	78	100	156	4
410	Winnipeg, Canada	760	...	40	1925	...	...	...	...	10.1	13.8	20.0	27.2	...	...	...	...	14.6	...	50.5	69	100	126	4
411	Prince Albert, Canada	1 432	...	41	1925	...	...	...	...	8.0	7.4	15.8	29.9	...	...	...	...	14.8	...	50.8	47	100	189	4
412	Edmonton, Canada	2 158	...	43	1925	...	...	...	...	5.7	11.2	17.1	27.8	...	...	...	...	15.6	...	44.0	47	100	152	4
413	Calgary, Canada	3 389	...	41	1925	...	...	...	...	5.9	7.9	16.1	34.6	...	...	...	...	13.8	...	55.3	49	100	215	4
414	Barkerville, Canada	4 180	...	38	1925	...	...	...	...	6.3	20.6	27.6	49.5	...	...	...	...	8.9	...	39.3	68	100	168	4
415	Victoria, Vancouver, Canada	228	...	35	1925	...	...	...	...	13.0	20.7	30.4	51.0	...	...	...	...	19.0	...	43.0	68	100	168	4
416	Sitka, Alaska	85	45	64	1921	...	...	...	5.2	23.0	52.2	84.0	103.0	0.5	...	...	...	12.0	5.9	20.1	72	100	142	2; 20
417	Juneau, Alaska	94	16	36	1925	0.11	0.19	0.33	5.8	20.5	44.8	81.6	106.5	0.5	1.3	2.2	6.5	8.4	6.7	23.2	55	100	139	2; 6
418	Dawson, N. W., Canada	1 052	...	24	1925	...	...	...	...	6.2	9.2	14.4	17.7	...	...	...	...	9.2	...	29.2	64	100	123	4
419	Eagle, Alaska	838	20	25	1922	...	...	...	1.9	4.7	9.0	20.9	13.5	1.1	...	...	...	18.2	17.6	42.5	58	100	126	2; 4
420	Fairbanks, Alaska	500	11	18	1925	...	...	...	1.5	3.2	6.3	11.5	16.7	1.3	...	...	...	14.4	12.0	39.3	68	100	160	2; 4
421	Tanana, Alaska	223	18	25	1925	...	...	...	2.1	5.8	7.8	12.6	17.7	0.9	...	...	...	18.8	16.7	40.1	82	100	141	2; 4
422	Nome, Alaska	26	15	25	1925	...	...	...	2.1	3.4	9.5	16.2	25.3	0.9	...	...	...	15.3	13.2	48.4	61	100	168	4
423	Rodiak, Alaska	18	14	33	1923	...	...	...	4.5	17.8	43.6	80.7	93.6	...	...	...	...	12.2	7.4	29.5	72	100	138	4; 2
424	St. Paul Island, Alaska	20	...	21	1921	...	...	...	...	9.2	22.5	31.5	47.1	0.5	...	...	...	11.4	...	34.4	60	100	134	4
425	Dutch Harbor, Alaska	16	15	18	1922	...	...	...	4.1	20.2	40.9	62.7	74.1	0.6	...	...	...	13.4	6.6	33.3	65	100	116	2; 4
426	Midway Island	...	9	9	...	...	...	...	6.3	...	...	46.3	...	...	...	...	...	11.2	13.6	...	...	100	...	7
427	Kealia, Kauai, Hawaiian Islands	14	16	19	1918	...	...	...	8.7	24.3	16.7	40.8	62.5	...	...	...	...	16.7	21.3	59.5	41	100	153	2
428	Honolulu, Oahu, Hawaiian Islands	58	23	54	1927	...	...	...	12.5	24.8	11.3	30.9	55.9	0.6	...	...	...	12.4	24.3	81.0	37	100	180	2; 1
429	Lualaha, Oahu, Hawaiian Islands	1 111	16	34	1918	...	...	...	10.7	16.9	9.4	30.3	205.0	0.4	...	...	...	8.3	6.7	23.0	60	100	126	2; 7
430	Kaanapali, Maui, Hawaiian Islands	12	...	20	1918	...	...	...	...	17.0	6.0	20.9	91.4	...	...	...	...	20.9	...	91.4	27	100	168	2
431	Puu Kukui Maui, Hawaiian Islands	5 000	7	7	1918	...	...	...	...	26.0	232.0	370.1	562.0	...	...	...	...	9.5	...	25.9	63	100	152	2
432	Waiahuk Mill, Maui, Hawaiian Islands	175	16	27	1916	...	...	...	6.2	23.9	8.7	27.8	54.5	0.7	...	...	...	14.6	22.2	85.8	31	100	192	2
433	Keanae Valley, Maui, Hawaiian Islands	1 000	16	14	1918	...	...	...	21.9	71.0	129.0	253.6	397.3	0.3	...	...	...	11.4	8.6	27.9	51	100	156	2
434	Kawaiinui, Hawaii, Hawaiian Islands	4 080	...	12	1918	...	...	...	...	28.0	29.0	50.8	75.5	...	...	...	...	10.4	...	30.2	58	100	182	2
435	Waimea, Hawaii, Hawaiian Islands	2 700	...	28	1918	...	...	...	...	27.3	24.4	43.5	89.1	...	...	...	...	11.5	...	62.8	51	100	184	2
436	Hinuuia, Hawaii, Hawaiian Islands	6 835	...	28	1918	...	...	...	...	26.0	17.8	39.1	84.7	...	...	...	...	13.1	...	53.3	49	100	168	2
437	Hilo, Hawaii, Hawaiian Islands	40	16	36	1918	...	...	...	16.5	58.6	91.2	139.4	194.2	0.4	...	...	...	10.9	10.4	42.0	85	100	139	2; 7
438	Volcano House, Hawaii, Hawaiian Is.	4 005	...	17	1918	...	...	...	...	34.0	43.0	78.3	127.0	...	...	...	...	9.6	...	33.1	72	100	165	2
439	Kapapala Ranch, Hawaii, H. I.	2 150	...	33	1918	...	...	...	...	34.4	33.9	58.1	114.0	...	...	...	...	12.3	...	59.1	44	100	197	2
440	Hilea, Hawaii, Hawaiian Islands	220	...	29	1918	...	...	...	...	25.0	19.4	52.1	97.0	1.3	...	...	...	13.8	...	68.5	28	100	243	2
441	Fanning Island	...	...	13	1917	...	...	...	...	...	47.4	118.2	208.8	...	...	...	...	...	...	...	...	100	177	7
442	Malden Island	...	23	...	31	1919	...	...	...	25.7	3.9	28.6	93.6	1.4	...	...	...	16.6	...	90.0	14	100	627	4
443	Papeiti, Society Island	...	20	...	35	1909	...	...	...	42.4	12.3	55.5	130.3	...	...	...	...	14.8	...	76.5	22	100	235	7
444	Rarotanga, Cook Island	...	20	...	20	...	...	...	...	38.2	46.0	82.7	116.6	...	...	...	...	14.2	...	46.3	55	100	141	7
445	Apia, Samoan Islands	...	9	29	31	1920	...	...	...	10.2	40.0	65.8	107.2	172.7	...	...	...	15.5	9.5	37.3	64	100	161	4; 7
446	Pago Pago, Samoan Islands	...	...	21	22	...	...	...	...	20.6	...	...	186.2	...	...	...	...	7.8	10.2	...	...	100	...	7
447	Delanauna, Fiji Islands	...	70	10	40	...	...	...	...	14.8	...	...	92.9	...	0.7	...	...	17.5	16.1	...	...	100	...	7
448	Wainunu, Fiji Islands	...	...	...	39	...	...	...	...	...	...	...	148.2	...	...	...	...	...	...	...	...	100	...	7
449	Suva, Fiji Islands	...	44	16	34	...	...	...	15.9	37.3	73.0	112.4	169.6	0.4	...	...	...	8.5	14.1	33.2	65	100	151	7
450	Auckland, New Zealand	...	125	...	71	1923	...	...	...	14.1	26.3	43.9	74.1	...	...	...	...	6.9	...	32.1	60	100	169	4
451	Wellington, New Zealand	...	10	...	62	1923	...	...	...	12.9	28.5	48.0	67.7	...	...	...	...	8.5	...	26.0	69	100	140	4
452	Christchurch, New Zealand	...	25	...	47	1923	...	...	...	9.1	19.8	25.1	35.3	...	...	...	...	10.4	...	31.8	54	100	140	4
453	Hokitika, New Zealand	...	9	...	45	1923	...	...	...	29.1	88.2	116.0	154.4	...	...	...	...	8.3	...	25.1	76	100	133	4
454	Noumea, New Caledonia	...	30	...	55	1914	...	...	...	19.1	19.6	43.5	73.8	0.8	...	...	...	13.4	...	44.0	45	100	174	7
455	Col D'Amieu, New Caledonia	1 150	...	...	5	1914	...	...	...	...	...	78.3	...	0.6	...	...	...	...	...	...	...	100	...	7
456	Tulagi, Solomon Islands	...	7	...	21	1918	...	...	...	41.9	62.1	115.4	162.1	...	...	...	...	14.2	...	37.3	54	100	141	7
457	Ocean Island	...	85	12	12	1916	...	...	...	9.7	...	80.8	...	...	...	...	...	11.2	8.3	...	...	100	...	7
458	Nauru, Gilbert Island	...	26	16	16	1913	...	...	...	7.7	29.6	21.4	84.2	163.2	...	...	...	13.1	9.1	35.2	25	100	194	7
459	Jaluit, Marshall Islands	...	15	17	17	1913	...	...	...	11.1	...	...	158.6	...	...	...	...	9.7	7.0	...	...	100	...	7
460	Ujelang Island	...	30	17	17	1913	...	...	...	11.7	...	...	77.4	...	...	...	...	12.4	15.2	...	...	100	...	7
461	Guam Island	...	67	17	17	1922	...	...	...	10.5	...	...	91.1	...	...	...	...	15.6	11.5	...	...	100	...	7
462	Yap, Caroline Islands	...	104	20	20	...	...	...	...	12.6	45.8	72.9	119.1	152.0	...	...	...	13.2	10.6	38.5	61	100	128	7
463	Namatanai, New Ireland	...	...	7	7	1913	...	...	...	8.8	...	...	110.6	...	...	...	...	14.1	8.0	...	...	100	...	7
464	Funiolo, New Britain	...	...	19	19	1913	...	...	...	7.5	28.3	42.2	84.0	133.2	0.6	...	...	6.3	8.9	30.0	50	100	158	7
465	Peterhafen, French Islands	...	...	10	10	1913	...	...	...	11.0	...	...	130.2	...	...	...	...	13.5	8.4	...	...	100	...	7
466	Port Moresby, New Guinea	...	127	...	29	1920	...	...	...	18.5	21.6	40.8	72.0	...	...	...	...	18.4	...	45.4	53	100	176	4
466-A	Crohamhurst, Australia	1 400	20	20	1913	...	...	...	35.7	107.4	24.7	80.1	201.7	...	...	...	...	19.7	44.6	134.3	31	100	252	15
467	Brisbane, Australia	137	74	74	1925	...	...	...	18.3	40.4	16.2	45.3	88.3	...	...	...	...	13.8	40.5	95.3	36	100	185	29; 4
468	Sydney, Australia	138	67	85	1924	0.50	1.35	2.90	6.9	25.4	21.5	47.9	82.6	...	1.5	2.8	6.1	11.6	18.6	53.0	45	100	173	4; 29; 8
469	Melbourne, Australia	115	67	70	1925	...	...	...	8.4	7.9	15.4	25.0	42.4	...	...	...	...	9.6	13.7	40.8	41	100	197	29
470	Hobart, Tasmania	177	59	83	1925	...	...	...	5.0	10.2	13.4	23.8	43.4	...	...	...	...	7.7	21.0	42.7	59	100	152	15; 12
471	Adelaide, Australia	140	57	87	1925	...	...	...	5.6	8.4	11.4	21.2	30.9	...	...	...	...	14.9	20.5	40.6	50	100	146	29; 4
471-A	Peechyarrina, Australia	...	...	...	21	1903	...	...	...	4.3	4.2	5.5	8.8	...	...	...	...	12.0	...	98.5	27	100	196	15
472	Alice Springs, Australia	1 928	...	...	51	1924	...	...	...	11.3	4.3	11.1	28.6	...	...	...	...	14.2	...	162.0	35	100	258	4
472-A	Nullarbor, Australia	...	...	...	24	1911	...	...	...	3.9	4.6	6.5	18.9	...	...	...	...	20.4	...	62.1	39	100	199	15
473	Perth, S. W. Australia	...	197	50	50	1925	...	...	...	3.0	12.8	9.2	36.0	46.7	...	...	...	20.4	11.5	37.7	59	100	138	29; 18
474	Carnarvon, W. Australia	...	...	...	15	...	...	...	...	...	...	6.8	...	...	...	...	...	...	...	...	...	...	...	18
475	Darwin, N. W. Australia	...	...	23	24	1924	...	...	...	11.7	27.9	40.6	61.8	87.2	...	...	...	25.8	18.0	48.1	66	100	141	4; 15
476	Kupang, East Indies	...	48	...	45	1923	...	...	...	42.3	30.4	58.2	87.6	...	...	...	...	27.1	...	72.9	52	100	159	4
477	Kajoemas, East Indies	3 050	...	...	36	1923	...	...	...	40.3	70.4	99.4	154.0	...	...	...	...	17.9	...	40.6	71	100	155	4

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.		Mean month.	Maximum.	
		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.		24 hours.	Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
367	2.01	7.3	3.17	7.2	2.64	10.7	2.24	12.4	1.14	6.5	0.91	5.4	0.58	5.1	0.94	5.2	1.57	6.4	2.32	5.7	2.09	8.3	2.09	9.4
368	2.01	12.3	2.62	7.5	2.22	8.9	1.76	14.3	3.06	13.6	2.63	8.3	2.20	6.3	2.48	10.9	2.99	13.7	2.65	6.8	2.77	8.5	2.68	6.6
369	2.13	11.2	4.23	10.5	3.48	8.5	1.07	8.0	2.87	9.6	2.52	6.8	2.29	3.3	0.49	3.1	0.89	4.5	3.27	7.9	3.19	8.5	3.86	12.5
370	1.41	5.2	2.68	5.3	0.78	1.9	0.66	4.2	1.00	3.1	0.39	1.2	0.09	0.6	0.05	0.5	0.14	3.0	0.49	3.4	0.63	3.0	2.44	6.3	3.72	7.5	3.07	8.5	1.30	5.0	0.45	1.7		
371	0.94	3.5	1.79	3.5	0.59	1.9	0.66	3.2	0.51	2.0	0.55	1.7	0.43	1.9	0.47	3.4	0.59	1.7	0.55	1.9	0.70	2.4	0.91	5.9	1.30	4.7	3.00	10.7	4.65	12.4						
372	7.05	12.8	5.82	15.2	4.25	12.0	5.20	9.6	4.61	13.1	2.72	6.9	2.20	5.5	1.54	5.6	3.11	9.0	5.47	13.1	5.91	13.5	6.19	18.6										
373	5.47	35.7	5.12	15.2	4.25	12.0	5.20	9.6	4.61	13.1	2.72	6.9	2.20	5.5	1.54	5.6	3.11	9.0	5.47	13.1	5.91	13.5	6.19	18.6										
374	2.68	6.7	2.83	7.2	2.23	10.6	4.47	14.6	3.51	12.0	3.15	14.3	2.49	9.0	3.02	15.6	4.80	9.2	5.63	16.2	4.96	13.5	5.59	11.5										
375	6.71	14.4	5.82	10.7	4.48	9.2	3.07	7.7	4.10	14.7	3.98	10.1	2.56	7.0	3.27	20.7	10.75	26.7	12.44	25.0	14.00	35.7	14.84	12.5										
376	16.89	31.0	15.39	43.4	14.45	34.1	12.13	59.1	8.45	16.1	2.26	12.2	1.68	6.4	1.75	11.2	2.58	14.9	1.02	4.0	1.06	4.5	1.16	8.4										
377	4.96	14.0	4.59	13.1	6.45	27.6	8.50	32.0	11.20	32.0	11.20	12.4	2.13	10.4	1.10	6.6	0.67	5.5	2.56	5.9	2.52	5.3	5.48	10.9										
378	2.66	10.2	2.27	6.1	2.01	11.7	2.66	13.4	2.77	9.6	3.32	7.2	4.72	10.1	4.13	7.0	2.35	5.5	2.56	5.9	2.52	5.3	5.48	10.9										
379	3.85	18.3	2.93	15.3	2.14	30.9	12.00	16.8	8.90	14.0	5.55	7.2	5.06	10.1	4.13	7.0	2.35	5.5	2.56	5.9	2.52	5.3	5.48	10.9										
380	10.90	13.9	12.18	15.3	14.15																													
381	14.18		12.08			22.7	15.55			19.80			14.80			22.0	9.89	15.5	6.58	14.8		1.10	7.6		1.34			4.65			10.53				
382	7.85	18.2	4.63	13.4	7.16		6.01	13.6	3.14	24.4	11.68								2.59	6.48		3.08			2.95	8.9	6.68		20.7	11.14		11.53			26.6	
383	2.75	9.3	11.7	1.52	1.5	6.3	1.52	1.5	7.5	1.84	2.5	8.0	3.64	2.3	11.6	7.95	2.0	16.3	8.85	2.5	17.5	9.63	2.7	17.4	7.37	2.9	12.7	6.65	3.2	14.6	7.00	2.2	15.1	4.66	1.8	11.9
384	7.32			4.67			4.20			5.43			13.75			13.99			12.55			8.71			9.43			13.29			11.53				
385	4.43	18.6	9.8	2.78	1.4	7.2	2.74	1.2	6.8	2.28	1.3	5.9	4.59	3.5	10.8	8.32	2.9	19.4	9.79	5.7	17.8	9.58	3.4	19.1	8.26	3.9	12.6	7.65	4.3	14.2	8.39	4.3	20.9	7.27	2.9	18.2
386	3.37	14.4	10.2	2.22	0.9	12.9	1.92	1.8	5.1	1.65	2.6	6.7	2.69	1.1	12.3	4.34	2.0	15.5	5.13	3.2	14.5	7.41	3.8	17.8	7.77	4.6	22.2	7.63	4.4	28.4	6.48	2.4	20.4	4.23	2.5	11.7
387	4.45			3.56			2.16			2.42			5.45			7.25			9.08			9.83			9.47			6.83			8.12			6.17		
388	9.68			6.79			4.79			6.29			6.29			11.59			13.48			13.54			11.11			9.95			10.57			12.07		
389	2.29	5.2	2.04	7.1	1.24	3.7	2.97	17.2	4.38	14.3	4.63	13.7	3.47	12.9	4.25	18.7	5.57	25.3	5.88	18.0	3.67	12.9												
390	2.71	1.4	1.87	1.6	1.22	2.2	2.12	2.8	3.09	2.4	3.19	1.3	3.09	2.7	3.38	4.6	5.44	2.9	4.39	2.5	4.13	6.7	4.25	8.0	4.39	17.6	5.88	18.0	3.67	12.9				
391	4.07	3.1	12.4	2.83	4.8	7.8	2.67	2.8	7.2	4.38	6.7	13.2	5.13	4.8	24.0	5.30	5.2	12.2	5.73	4.9	11.0	5.86	6.3	10.4	6.06	4.3	13.7	5.53	2.3	10.2	6.91	5.5	13.3	5.72	3.0	15.4
392	3.97	3.4		4.75	5.4		4.01	2.6		7.96	7.0		8.68	3.6		6.53	4.1		7.18	9.1		6.70	9.0		32.78	5.1		25.8	5.0		4.95	3.8			
393	2.07	1.5	1.20	0.6			2.75	2.8		3.68	6.2		5.06	5.0		5.76	2.4		7.24	5.4		6.67	5.4		7.22	5.1		7.29	7.4		4.25	2.6		2.29	1.0	
394	1.23	2.9	6.4	2.51	1.6	10.9	2.69	3.5	12.4	6.46	3.9	14.2	9.35	4.5	13.7	6.96	2.2	13.7	2.69	2.4	5.7	5.27	4.6	10.7	7.29	5.8	21.7	6.57	3.0	14.6	3.44	6.1	12.7	2.29	1.0	4.9
395	4.38	6.3		2.89	3.3		2.15	4.7		3.53	5.5		4.16	3.8		4.00	3.3		6.21	8.3		3.40	4.9		5.25	5.8		4.26	3.6		6.16	4.8		6.11	3.6	
396	0.92	1.5		4.07	6.8		1.88	1.4		5.98	4.4		9.35	8.9		6.06	3.5		4.42	5.6		1.54	4.4		4.42	6.4		6.86	2.3		6.16	3.6		1.55	2.6	
397	1.47	3.1		4.88	5.0		5.98	4.4		9.35	8.9		4.16	3.8		4.00	3.3		6.21	8.3		3.40	4.9		5.25	5.8		4.26	3.6		6.16	4.8		6.11	3.6	
398	17.48	10.7		12.92	11.0		3.83	5.5		4.16	3.8		4.00	3.3		6.21	8.3		3.40	4.9		5.25	5.8		4.26	3.6		6.86	2.3		6.16	4.8		6.11	3.6	
399	1.47	3.4		0.87			1.88	1.4		5.98	4.4		9.35	8.9		6.06	3.5		4.42	5.6		1.54	4.4		4.42	6.4		6.86	2.3		6.16	4.8		6.11	3.6	
400	1.47	3.4		2.16	1.5		5.98	4.4		9.35	8.9		4.16	3.8		4.00	3.3		6.21	8.3		3.40	4.9		5.25	5.8		4.26	3.6		6.16	4.8		6.11	3.6	
401	2.90	6.7	11.8	1.81	3.4	6.2	1.69	1.7	9.6	2.13	2.2	6.0	3.53	2.2	10.4	5.39	1.3	13.7	3.70	1.7	11.7	1.58	3.1	3.47	2.5	16.4	6.59	4.2	20.1	7.55	6.5	20.5	6.34	5.0	23.4	2.85	7.4	10.4
402	2.16	4.5	9.7	1.61	2.7	7.0	1.53	2.5	5.3	2.51	2.9	9.5	5.89	7.4	17.8	6.00	5.4	15.3	5.83	4.4	14.4	6.59	6.4	20.1	7.55	6.5	20.5	5.95	5.0	17.7	5.01	6.0	11.4	4.78	4.6	10.6
403	4.46	3.8	9.7	4.60	3.3	10.4	4.74	3.9	10.0	4.10	10.7	13.3	4.54	5.5	13.1	4.49	8.0	11.0	4.53	3.3	18.2	5.40	3.1	21.3	5.19	6.6	16.3	4.92	13.9	5.69	8.7	5.51	9.7	
404	5.39	8.5	4.69	8.2	4.38	8.5	3.54	5.6	3.40	6.5	3.40	10.2	3.63	11.3	3.65	17.9	3.37	8.7	4.25	10.4	4.17	6.0	3.76	7.2										
405	3.57	7.6	2.89	6.4	2.25	8.5	2.89	6.2	2.62	5.5	2.65	7.8	3.75	7.7	3.72	8.8	3.76	11.4	5.01	8.1	2.87	6.0	2.78	6.7										
406	5.37	11.4	4.96	12.8	4.57	8.1	1.96	4.0	2.91	6.7	3.26	8.6	3.76	7.2	3.31	7.9	3.52	7.8	3.31	5.4	2.58	5.6	2.44	4.9										
407	3.69	6.2	2.58	6.3	2.53	8.5	2.55	5.4	2.76	9.1	2.70	5.8	2.55	6.1	2.05	7.0	2.10	4.7	2.29	5.5	1.35	5.7	1.07	3.0	0.91	4.0								
408	3.69	6.2	3.16	6.3	2.53	8.5	2.55	5.4	2.76	9.1	2.70	5.8	2.55	6.1	2.05	7.0	2.10	4.7	2.29	5.5	1.35	5.7	1.07	3.0	0.91	4.0								
409	2.52	3.4	0.78	2.5	1.16	2.9	1.42	5.0	1.94	4.4	2.92	10.1	3.31	7.1	2.10	4.7	2.29	5.5	1.35	5.7	1.07	3.0	0.91	4.0										
410	0.94	2.0	0.62	2.1	0.89	2.7	0.91	2.4	1.70	4.9	2.74	4.6	2.23	5.3	2.35	8.0	1.42	3.3	0.92	1.9	0.99	3.1	0.66	2.2										
411	0.78	2.0	0.62	2.1	0.89	2.7	0.91	2.4	1.70	7.7	3.13	8.5	3.38	11.1	2.43	7.7	1.25	4.0	0.61	2.2	0.69	2.1	0.82	2.2										
412	0.88	2.5	0.69	1.9	0.82	2.4	0.84	2.2	2.22	4.3	3.49	7.8	3.20	8.5	2.67	4.5	2.57	5.6	5.42	21.0	5.74	21.0												
413	2.34	9.3	2.62	5.2	2.62	6.5	1.32	5.4	1.12	2.6	0.84	3.9	7.14	4.13	3.10	9.7	13.4	3.21	10.15	5.25	3.04	17.4	24.8	5.24	4.21	5.8	8.40	4.0	16.5	7.36	2.7	13.3				
414	2.34	9.3	2.62	5.2	2.62	6.5	1.32	5.4	1.12	2.6	0.84	3.9	7.14	4.13	3.10	9.7	13.4	3.21	10.15	5.25	3.04	17.4	24.8	5.24	4.21	5.8	8.40	4.0	16.5	7.36	2.7	13.3				
415	7.82	3.16	6.50	2.58	5.68	9.6	5.34	2.6	11.4	5.52	4.4	13.1	3.70	1.7	11.7	1.58	3.1	3.47	2.5	16.4	6.59	4.2	20.1	7.55	6.5	20.5	6.34	5.0	23.4	2.85	7.4	10.4				
416	7.82	3.16	6.50	2.58	5.68	9.6	5.34	2.6	11.4	5.52	4.4	13.1	3.70	1.7	11.7	1.58	3.1	3.47	2.5	16.4	6.59	4.2	20.1	7.55	6.5	20.5	6.34	5.0	23.4	2.85	7.4	10.4				
417	5.64	2.20	5.04	1.6	5.19	8.9	5.34	2.6	11.4	5.52	4.4	13.1	3.70	1.7	11.7	1.58	3.1	3.47	2.5	16.4	6.59	4.2	20.1	7.55	6.5	20.5	6.34	5.0	23.4	2.85	7.4	10.4				
418	0.83	2.0	0.76	1.6	0.54	1.8	0.62																											
419	0.46	1.4	0.35	1.2	0.44	1.2	0.41	2.2	0.41	1.0	0.32	1.0	0.32	2.9	1.48	1.0	3.1	1.78	0.9	2.6	1.06	1.9	4.7	1.27	1.0	3.4	0.82	2.8	3.0	0.49	0.5	1.3	0.49	0.7	1.1	0.66
420	0.74	0.7	0.35	0.5	0.35	0.9	0.41	1.0	0.32	1.0	0.32	2.9	1.48	1.0	3.1	1.78	0.9	2.6	1.06	1.9	4.7	1.27	1.0	3.4	0.82	2.8	3.0	0.49	0.5	1.3	0.49	0.7	1.1	0.66
421	0.74	0.7	0.35	0.5	0.35	0.9	0.41	1.0	0.32	1.0	0.32	2.9	1.48	1.0	3.1	1.78	0.9	2.6	1.06	1.9	4.7	1.27	1.0	3.4	0.82	2.8	3.0	0.49	0.5	1.3	0.49	0.7	1.1	0.66
422	0.74	0.7	0.35	0.5	0.35	0.9	0.41	1.0	0.32	1.0	0.32	2.9	1.48	1.0	3.1	1.78	0.9	2.6	1.06	1.9	4.7	1.27	1.0	3.4	0.82	2.8	3.0	0.49	0.5	1.3	0.49	0.7	1.1	0.66
423	0.74	0.7	0.35	0.5	0.35	0.9	0.41	1.0	0.32	1.0	0.32	2.9	1.48	1.0	3.1	1.78	0.9	2.6	1.06	1.9	4.7	1.27	1.0	3.4	0.82	2.8	3.0	0.49	0.5	1.3	0.49	0.7	1.1	0.66
424	2.54	7.7	1.99	3.3	1.53	5.2	1.62	3.2	2.12	2.2	3.39	2.0	6.1	5.04	2.6	7.5	2.71	1.9	7.1	2.31	1.3	9.0	2.1	5.6	2.18	0.8	8.40	4.1	20.2	6.17	3.4	13.0	7.23	3.0	15.6	
425	2.54	7.7	1.99	3.3	1.53	5.2	1.62	3.2	2.12	2.2	3.39	2.0	6.1	5.04	2.6	7.5	2.71	1.9	7.1	2.31	1.3	9.0	2.1	5.6	2.18	0.8	8.40	4.1	20.2	6.17	3.4	13.0	7.23	3.0	15.6	
426	4.19	3.4	6.98	3.3	4.24	3.7	5.08	5.5		3.60	6.1		3.38	4.0		3.08	4.1		3.54	3.1		5.01	5.8		5.20	6.3		1.74	1.4		3.61	2.0			
427	4.10	5.8	15.7	3.96	3.0	21.4	6.84	5.2	24.3	2.50	1.9	9.6	2.43	2.6	7.8	1.92	1.9	2.9	2.00	1.8	4.3	1.97	1.4	4.5	3.02	3.8	4.3	3.16	3.0	9.3	3.98	3.7	11.1	5.06	4.2	14.6
428	3.82	4.5	18.4	3.89	2.5	24.8	5.71	3.5	25.9	2.12	1.7	9.6	2.43	2.6	7.8	1.92	1.9	2.9	2.00	1.8	4.3	1.97	1.4	4.5	3.02	3.8	4.3	3.16	3.0	9.3	3.98	3.7	11.1	5.06	4.2	14.6
429	12.32	6.0	46.4	9.68	10.3	35.9	15.59	6.0	34.3	16.03	3.5	26.0	11.25	2.6	41.0	4.48	1.9	4.23	5.7	0.56	8.1	0.68	38.0	23.5	31.0	6.2	55.0	32.48	7.5	85.0				
430	4.72	17.5	3.24	10.0	2.24	60.0	46.01	92.0	20.0	45.0	33.51	6.5	9.00	3.5	0.42	0.7	1.1	1.03	1.5	3.1	0.89	1.5	2.4	1.42	1.1	4.9	3.02	3.6	21.0	6.34	4.2	16.0				
431	4.72	17.5	3.24	10.0	2.24	60.0	46.01	92.0	20.0	45.0	33.51	6.5	9.00	3.5	0.42	0.7	1.1	1.03	1.5	3.1	0.89	1.5	2.4	1.42	1.1	4.9	3.02	3.6	21.0	6.34	4.2	16.0				
432	4.14	5.2	9.0	4.58	2.3	9.2	3.91	3.2	15.0	2.15	1.7	9.1	2.21	1.0	11.0	2.59	2.0	12.0	2.50	2.7	9.0	2.45	1.9	7.0	2.59	1.6	9.0	2.32	1.7	13.1	4.26	2.7	17.2	5.59	2.6	17.2
433	14.97	6.7	28.9	17.04	11.6	37.5	21.72	11.0	63.1	28.83	13.7	71.0	21.00	4.1	59.8	19.87	5.0	35.7	20.50	7.9	44.7	25.78	10.8	56.1	19.61	7.3	45.5	15.94	6.9	32.7	22.15	7.9	66.0	21.42	7.2	49.4
434	18.17	7.1	36.5	14.05	11.2	41.5	24.90	9.2	66.0	49.23	24.90	42.5	28.67	5.3	120.30	32.45	7.4	3.25	10.4	2.9	7.0	2.61	8.0	3.52	2.9	9.2	3.52	2.9	16.2	5.11	3.2	19.5	6.13	3.2	14.3	
435	18.17	16.5	4.04	11.2	4.98	37.2	2.66	5.5	3.04	2.5	1.03	5.2	1.74	1.0	3.85	1.64	7.5	2.14	64.7	5.38	15.0	2.14	64.7	5.38	15.0	2.14	64.7	5.38	15.0	2.14	64.7	5.38	15.0	2.14	64.7	5.38
436	2.43	5.3	38.6	11.47	7.0	32.5	15.14	8.2	58.6	13.44	16.2	58.0	9.85	10.7	3.88	10.2	5.67	28.6	3.58	12.7	3.66	10.4	4.31	13.1	7.14	36.4	5.28	19.3	29.9					
437	9.93	5.6	40.1	6.94	3.1	30.4	9.15	2.9	40.0	15.8	3.4	37.0	5.86	10.7	3.88	10.2	5.67	28.6	3.58	12.7	3.66	10.4	4.31	13.1	7.14	36.4	5.28	19.3						
438	6.82	11.3	6.04	31.5	9.15	29.0	4.03	15.8	3.46	6.6	1.04	3.9	1.72	8.2	2.81	11.9	2.05	3.0	0.97	9.6	0.75	5.7	0.80	8.2										
439	3.70	16.0	4.12	19.7	4.99	29.0	2.83	12.5	2.79																									
440	2.72									13.99			11.49			11.24			5.71			3.99			4.98			3.70			12.99				
441	10.77		9.72				12.54			17.04																								
442	4.06	19.5	2.00	14.8	4.74	25.7	4.63	19.1	4.24	16.4	1.94	12.5	2.01	10.1	1.63	5.6	0.85	3.0	0.97	9.6	0.75	5.7	0.80	8.2										
443	8.21		7.45		6.94			4.34		3.33			2.72			1.94			1.63			2.29			3.52			5.18			8.01				
444	9.80		10.85		11.80			8.25		6.43			4.13			3.69			4.55			4.54			4.75			6.48			7.47				
445	16.86	6.5	40.0	16.53	5.7	50.1	13.41	6.3	44.2	10.16	9.6	20.0	5.54	4.8	17.7	5.14	4.8	12.4	2.72	2.1	6.4	3.50	6.1	15.4	4.99	8.0	14.4	6.32	4.3	12.4	9.00	3.8	20.2	13.59	10.2	26.4
446	21.92	10.2		23.50	13.5		19.06	7.8		17.67	8.6		15.27	9.0		14.38	16.5		9.00	10.6		7.66	16.0		12.87	15.9		15.76	7.5		19.33	16.5		18.93	13.0	
447	16.97	12.1		14.18	5.6		16.20	14.9		8.21	4.7		4.58	3.0		4.62	3.1		5.67	4.0		6.24	6.5		6.06	5.2		9.51	15.9		12.11	10.0			
448	18.95	5.1		17.45	7.4		18.49	8.1		11.26	6.9		10.16	7.4		6.15	5.2		4.60	3.9		8.24	6.3												
449	10.17	24.0		14.70	9.5		11.26	6.9		10.16	7.4		6.15	5.2		4.60	3.9		8.24	6.3														
450	2.65	7.4	3.03	14.1	3.04	8.7	3.25	9.9	4.61	10.7	4.55	11.2	5.02	10.1	4.20	10.6	3.63	7.4	3.63	10.2	3.26	7.7	2.85	8.8										
451	3.38	10.1	3.14	8.9	3.33	8.2	3.88	12.1	4.68	11.0	4.55	12.4	5.63	12.2	4.40	9.9	3.97	11.0	4.05	12.9	3.54	10.0	3.12	12.5										
452	2.33	20.5	7.26	17.9	9.68	29.1	21.9	9.82	22.1	6.62	8.0	21.6	6.55	17.3	9.39	17.9	9.20	18.7	12.80	26.7	10.61	20.1	10.62	22.4										
453	9.83																																	
454	3.79		4.77		5.81		5.13			4.50			3.63			3.69			2.46			2.06			2.35			2.64					
455	11.94		18.14		10.54		7.50			5.66			3.94			3.49			4.22			1.54			2.74			6.28					
456	13.44		16.18		16.45		8.24			7.50			5.66			6.26			6.86			2.41			7.74			7.91			8.43			10.05		
457	12.85	4.5		9.03	6.7		8.43	5.0		8.49	4.8		6.07	5.3		5.13	4.0		5.67	3.7		2.67	2.3		3.90	3.3		4.80	3.5		5.74	4.0		5.03	4.5	
458	8.64	4.6		11.06	7.1		6.46	6.0		5.49	3.9		5.72	4.7		4.98	3.0		6.74	4.0		6.06	4.0		6.51	4.3		5.27	4.4		6.78	6.5		10.42	5.8	
459	10.19	6.8		8.53	5.2		14.22	6.0		15.76	8

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record. For day or more.	Years of Record. For month and year.	Years of Record. Last year included.	Maximum Observed Precipitation. Number of Minutes. 5.	15.	60.	Maximum Observed Precipitation. Day of 24 hours.	Annual Precipitation. Month.	Annual Precipitation. Minimum.	Annual Precipitation. Mean.	Annual Precipitation. Maximum.	Observed Precipitation, in Percentages of Mean Annual Rainfall. Mean daily supply on days of rainfall.	Maximum During Short Periods. Number of Minutes. 5.	15.	60.	Mean for month of maximum fall.	Maximum. In 24 hours.	Maximum. In consecutive months.	Annual. Minimum.	Annual. Mean.	Annual. Maximum.	Main references.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
478	Pasuruan, Java	16	45	1923						22.4	28.9	51.5	80.2					19.3		45.4	56	100	175	4
479	Batavia, Java	23	60	1923						32.4	45.9	72.1	96.8					18.1		44.8	64	100	134	4
480	Medan, East Indies	66	45	1923						17.2	62.2	81.4	103.7					11.1		31.1	76	100	127	4
481	Padang, Sumatra	3	45	1923						36.3	136.4	174.9	205.5					11.4		30.7	78	100	116	4
482	Kuta-Raja, Sumatra	23	45	1923						22.7	47.0	65.0	86.5					10.4		35.0	72	100	133	4
483	Singapore, Malay										32.7	92.0	158.7								38	100	173	12
484	Pontianak, Borneo	10	45	1923						33.9	99.3	126.1	167.1					12.0		26.8	79	100	132	4
485	Sandakan, Borneo	105	33	1920						61.1	57.8	119.7	177.6					15.4		51.0	48	100	148	4
486	Menado, East Indies	13	45	1923						47.4	62.6	103.7	158.9					17.6		45.6	60	100	153	4
487	Amboina, East Indies	3	45	1923						42.2	59.0	134.8	242.7					18.3		46.1	44	100	160	4
488	Zamboanga, Philippine Islands	10	17	19	1919				6.3	13.1	18.0	39.1	60.0					13.0	16.1	33.5	46	100	154	28; 15
489	Iloilo, Philippine Islands	25	17	25	1922				7.9	25.3	45.3	88.4	127.5					19.8	9.2	64.0	53	100	148	28; 15
490	Legaspi, Philippine Islands	31	17	20	1922				11.7	44.5	48.6	127.0	202.3					15.7	9.2	35.0	34	100	159	28; 4
491	Manila, Philippine Islands	56	37	62	1926			2.56	13.2	28.1	35.7	80.0	154.4				3.2	20.4	16.5	97.6	45	100	193	4; 28; 15
492	Baguio, Philippine Islands	4 959	17	18	1919			3.58	46.6	133.0	125.7	182.0	355.6				1.9	22.1	25.3	73.0	69	100	196	28; 15
493	Aparri, N. Philippine Islands	20	17	27	1922				34.0	11.3	29.2	46.0	87.3	138.1				9.7	12.9	44.9	53	100	135	4; 28
494	Kashoryo, Formosa	1 378	12	1918						68.7	184.8	282.5	331.0					9.3		24.3	65	100	117	15; 28
495	Taihoku, Formosa	39	22	24	1920				11.3	37.0	93.0	83.5	110.0					14.0	13.9	44.4	+76	100	122	4; 15
496	Nagasaki, Japan	435	42	1920						38.8	40.5	76.9	100.3					17.6		50.5	53	100	139	4
497	Kioto, Japan	160	40	1920						24.7	44.5	61.9	81.1					12.2		40.0	76	100	132	4
498	Tokyo, Japan	70	45	1920						22.7	34.2	55.7	86.0					14.0		41.0	63	100	145	4
499	Miyako, Japan	99	37	1920						22.7	34.2	55.6	80.7					12.6		41.0	63	100	152	4
500	Nemuro, Japan	39	37	1920						11.9	25.3	37.5	57.5					13.5		31.5	46	100	152	4
501	Vladivostok, Siberia	98	14	1890						13.8	9.4	19.5	32.6	1.6				17.8		70.8	48	100	172	30; 31; 12
502	Chemulpo, Korea	222	16	1920						17.9	25.0	37.4	54.7					22.9		48.0	67	100	146	4
503	Peking, China	132	34	33	1882				9.9	19.8	9.5	24.6	41.9	1.3				24.2	40.3	80.7	39	100	170	20
504	Zi-Ka-Wei, China	22		48	1920					19.4	27.9	45.1	62.5					16.1		42.0	62	100	139	4
505	Hong Kong, China	108		37	1920					48.5	45.8	90.0 94.3	110.7					13.8		58.0	54	100	142	18) 4]
506	Saigon, Indo-China	36	15	1920						23.7	61.9	77.2	107.0					14.4		30.8	80	100	139	4
507	Mergui, Burma	66	56	1920						62.7	132.0	162.3	213.0					18.6		38.6	81	100	131	4
508	Fort Blair, Andaman Island	79	53	1920						41.5	83.9	115.6	150.3					16.4		36.0	73	100	130	4
509	Moulmein, Burma	77	71	1920						32.7	69.0	99.6	143.4					22.9		38.0	65	100	146	4
510	Rangoon, Burma	18	51	1920						27.2	52.0	77.2	114.0					21.6		35.8	69	100	144	4
511	Akyab, Burma	20	62	1920						97.8	142.0	203.8	323.5					20.3		47.8	70	100	158	4
512	Mandalay, Burma	256	35	1920						24.2	16.8	35.1	49.7					12.4		40.4	48	100	142	4
513	Gauhati, N. E. India	199	73	1920						36.2	46.6	66.5	99.1					19.0		51.5	70	100	149	4
514	Shillong, N. E. India	4 920	65	1920						39.9	51.8	83.1	121.2					16.0		48.0	62	100	146	4
515	Cherrapunji, N. E. India	4 309	62	1920					40.0	366.0	262.6	456.9	905.1					15.5	8.8	90.1	64	100	198	4
516	Calcutta, N. India	21	92	1920				4.0		45.4	39.4	64.3	93.3					15.5	6.3	70.8	61	100	148	4; 6
517	Patna, E. India	183	73	1920						30.3	21.8	42.2	77.1					25.0		98.0	48	100	168	4
518	Nagpur, Central India	1 017	73	1920				3.0		24.8	14.4	46.4	65.3				7.8	28.7		53.5	31	100	141	4; 5
519	Waltair, India	15	58	1920						30.2	17.1	41.0	79.5					22.9		73.8	42	100	194	4
520	Masulipatam, India	20	58	1920						26.6	21.3	39.2	65.0					20.2		67.0	53	100	164	4
521	Madras, India	22	108	1920				0.94	25.0	42.8	18.4	45.6	88.4				1.9	27.5	50.0	86.3	27	100	178	4; 5
522	Trincomalee, Ceylon	99	52	1920						35.2	34.9	62.4	93.1					22.6		90.4	42	100	164	4
523	Nuwara Eliya, Ceylon	6 186	52	1920						25.0	68.4	92.5	131.5					12.8		37.6	74	100	142	4
524	Colombo, Ceylon	40	51	1920						15.3	45.7	80.1	122.7					16.6		44.0	67	100	174	4
525	Kodaikanal, S. India	7 686	21	1920						16.8	40.6	62.2	79.7					15.6		37.0	75	100	128	4
526	Cochin, S. India	9	63	1920						49.5	80.6	114.3	165.0					24.2		43.3	70	100	144	4
527	Mangalore, S. India	72	66	1920						10.4	89.4	128.9	162.3					24.7		54.7	69	100	162	4
528	Bangalore, S. India	3 021	85	1920						19.5	16.0	35.3	56.6					17.0		55.1	45	100	160	4
529	Belgaum, S. India	2 562	69	1920						42.2	28.7	49.2	73.4					20.1		64.5	61	100	159	4
530	Bombay, S. India	37	104	1920				3.0	16.0	59.0	33.4	72.4	114.9				4.1	23.5	22.1	81.6	46	100	159	4; 5
531	Ahmadabad, S. India	163	55	1920						37.5	4.8	28.9	54.5					39.2		78.1	17	100	177	4
532	Allahabad, S. India	312	74	1920						30.5	15.7	37.1	76.2					28.3		78.1	40	100	195	4
533	Jaipur, India	1 431	52	1920						21.8	4.7	23.9	55.3					30.5		91.0	20	100	233	4
534	Simla, India	7 232	89	1920						42.5	40.4	80.0	109.7					20.0		53.2	57	100	164	4
535	Leh, India	11 503	62	1920						4.0	0.4	2.0	9.0					9.0		125.0	12	100	284	4
536	Lahore, India	1 113	62	1920	0.90	2.00	4.50	9.0	20.4	6.2	20.1	37.9						25.0		101.6	31	100	206	4
537	Peshawar, N. W. India	1 113	62	1920					17.0	4.1	19.0	39.0						16.0		64.0	30	100	206	4
538	Quetta, Baluchistan	5 502	42	1920						7.5	3.9	9.3	21.6					19.5		80.8	42	100	233	4
539	Khelat, Baluchistan	6 921	41	1920						7.2	4.1	7.4	13.8					18.9		97.1	55	100	194	4
540	Hyderabad, N. W. India	96	55	1920					6.0	15.8	0.6	7.4	23.0					37.0	31.0	207.0	7	100	378	4; 5
541	Karachi, N. W. India	13	65	1920						10.6	0.6	7.4	28.0					38.3		251.8	7	100	378	4
542	Muscat, E. Arabia	20	27	1920						4.5	0.9	4.2	10.4	5.5				27.6		107.1	21	100	347	4
543	Jask, Persia	13	28	1920						4.8	0.2	4.5	7.7					17.8		106.7	4	100	162	4
544	Bushire, Persia	14	44	1920						12.9	0.0	10.4	26.6					23.7		126.0	6	100	256	4
545	Mohrd, Persia	3 104	28	1920						7.5	3.1	9.5	20.3					25.7		56.8	34	100	163	4
546	Tehran, Persia	4 260	28	1920						7.5	3.6	9.5	15.3					17.4		78.9	28	100	161	4
547	Baghdad, Mesopotamia	125	31	1920						2.5	7.9	1.3	7.1	22.3	3.7			19.6	37.2	111.1	19	100	314	4
548	Aden, S. Arabia	24	40	1920						5.6	0.0	1.9	8.6					26.6		366.7	9	100	478	4
549	Khartoum, Sudan	1 280	22	1922					3.0	7.6	1.3	5.7	14.6					45.6	61.5	130.0	23	100	253	4; 19
550	Deim Zubeir, Sudan	2 168	14	15	1920				4.4	23.9		45.7	78.5					17.0	11.8	52.3		100	172	19
551	Gore, Abyssinia	6 990	13	13	1920				3.5	17.0		75.7	92.6					15.8	4.6	23.5		100	122	19
552	Adis-Abeba, Abyssinia	8 006	13	23	1920				3.0	22.5		49.6	74.9					15.2	6.6	45.4		100	151	19
553	Entebbe, Uganda	3 847		25	1920					18.8	47.2	58.0	75.7					16.8		27.8	51	100	131	4
554	Dar-es-Salaam, E. Africa		249	20	1912					23.8	19.4	42.3	56.9					28.0		56.3	46	100	124	4
555	Zanzibar, E. Africa		56	30	1920					31.4	27.5	60.2	82.8					23.8		55.3	46	100	154	4
556	Seychelles Is., E. Africa			30	1920					31.4	66.5	94.8	132.8					17.9		33.3	70	100	140	4
557	Tananarive, Madagascar	4 600		34	1923					26.4	27.2	53.7	69.6					21.4		49.1	51	100	130	4
558	Salisbury, Rhodesia	4 860		28	1923					18.8	21.7	31.9	42 0					23.2		59.0	68	100	134	4
559	Bulawayo, Rhodesia	4 443		27	1923					13.4	12.1	23.6	36.8					21.9		56.8	51	100	156	4
560	Belvedere, S. Africa	5 000		17	1925								51.4									100		31
561	Johannesburg, S. Africa	5 925		27	1924					20.0	19.8	34.2	54.9	1.1				18.8		60.2	60	100	165	4
562	Kroonstad, S. Africa	4 600		45	1925							24.2	45.7									100		31
563	Alfred, S. Africa	4 352		42	1924					12.3	8.9	22.7	35.8					14.3		59.5	44	100	176	4
564	Kokstad, S. Africa	4 800		42	1924							27.4										100		31
565	Durban, S. Africa	265		39	1920					20.7	27.2	42.7	80.5	0.6				13.0		62.6	64	100	165	4
566	Port Elizabeth, S. Africa	181		59	1924					9.3	19.6	22.3	34.1	1.3				9.3		41.2	46	100	152	4
567	Cape Town, S. Africa	40		87	1924					13.1	19.0	23.7	37.0					15.1		53.3	69	100	162	4
568	Carnarvon, S. Africa	4 000		46	1925							8.1										100		31
569	Springbok, S. Africa	3 200		45	1925							3.3										100		31
570	O'Okiep, S. Africa	3 035		42	1924					4.3	2.1	6.7	11.7					16.8		64.0	31	100	174	4
571	Port Nolloth, S. Africa	25		41	1925							0.7										100		31
572	Bethany, S. W. Africa			14	1913						1.2	4.0	9.1					22.7		102.5	30	100	227	15
573	Hasuur, S. W. Africa			14	1913					7.1	0.3	7.6	25.6					29.6		93.6	4	100	338	15
574	Steinkopf, S. W. Africa	2 000		36	1925							3.3										100		31
575	Nitis, S. W. Africa	1 500		47	1925							3.3										100		31
576	Rietfontein, S. W. Africa	2 720		17	1925							7.1		4.3								100		31
577	Keetmanshoop, S. W. Africa	3 700		45	1925							6.7										100		31
578	Prieska, S. Africa	3 100		48	1925							8.6										100		31
579	Hopetown, S. Africa	3 604		46	1925							12.4										100		31
580	Kimberley, S. Africa	3 947		50	1923					9.0	8.6	12.4	31.1	1.6				17.4		53.5	55	100	146	4
581	Vryburg, S. Africa	3 880		33	1925							19.2		3.6								100		31
582	Mafeking, S. Africa	4 194		33	1925							22.9										100		31
583	Palapye Road, S. Africa			6								16.5										100		15
584	Tuan, S. W. Africa			25								12.6										100		15
585	Windhoek, S. W. Africa	5 461		22	1913					12.2	7.1	15.8	29.5					25.4		80.1	48	100	196	15
586	Swakopmund, S. W. Africa	23		15	1913					0.2	0.1	0.6	1.2					16.7		133.3	17	100	200	15
587	Waterberg, S. W. Africa	5 248		21	1913					20.2	7.3	21.1	41.6					26.3		96.0	35	100	197	15
588	Mossamedes, Angola			10								1.47										100		15
589	Loanda, Angola			42								12.4										100		15
590	Casenge, Angola			10								42.3										100		15
591	Camaxilho, Angola			4								52.6												

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
478	9.92		23.4	10.83		19.9	8.23		17.5	5.16		18.1	3.54		19.3	2.32		10.1	1.10		10.4	0.24		4.1	0.28		4.1	0.83		8.1	2.46		12.1	6.54		11.1
479	13.02		32.4	12.83		27.8	9.84		18.4	5.16		14.7	3.86		13.1	1.74		3.65	0.84		8.3	1.05		7.6	2.87		9.6	4.49		11.7	5.47		11.0	8.46		22.4
480	5.55		12.4	3.66		10.6	4.17		12.8	5.20		11.5	7.24		15.4	5.12		16.9	5.35		13.9	7.24		14.9	8.58		13.4	10.20		14.2	10.00		16.9	9.09		17.2
481	13.58		25.6	10.24		22.9	11.61		21.6	14.49		28.5	12.24		23.8	12.76		22.9	11.22		27.0	13.58		27.9	16.02		29.5	19.82		32.9	19.09		33.3			
482	5.91		13.6	2.50		9.8	3.50		8.8	4.21		14.0	6.06		16.5	3.62		9.0	4.25		16.5	4.45		15.0	6.77		22.7	7.20		15.9	7.24		19.3	8.27		20.7
483																																				
484	10.75		16.9	7.80		20.8	9.76		23.0	10.87		21.1	10.59		20.4	8.74		17.8	6.50		19.2	8.66		18.4	8.27		20.0	15.08		33.9	15.91		24.8	13.19		30.7
485	16.45		61.1	9.62		34.5	8.09		24.9	4.07		9.4	5.91		12.4	7.30		17.1	6.55		11.9	8.00		19.2	9.39		20.4	10.00		16.7	14.66		31.4	17.66		32.5
486	18.27		47.4	14.12		26.7	11.19		26.9	7.83		19.4	6.46		14.8	6.42		11.2	4.80		14.6	3.70		9.6	3.35		9.6	4.72		15.6	8.50		29.5	14.45		45.0
487	5.12		11.6	4.61		10.5	5.23		19.2	11.38		24.9	19.84		55.4	24.80		64.2	23.07		56.3	15.94		57.1	9.09		38.2	6.18		18.9	10.87		35.9	11.10		39.4
488	2.24	3.6	11.9	2.01	2.6	7.2	1.40	2.9	4.1	1.60	2.0	4.3	2.92	3.1	6.4	4.01	4.6	7.1	4.12	3.2	8.4	3.45	2.2	7.6	4.20	4.5	11.2	5.00	4.8	13.1	3.83	3.1	9.4	4.15	6.3	11.0
489	2.09	2.5	7.8	1.48	1.2	6.2	1.20	1.1	4.6	0.49	2.5	6.2	0.66	4.4	13.2	0.95	2.2	18.0	0.27	7.9	25.3	0.23	2.3	30.0	0.22	3.6	24.1	0.35	3.0	27.2	0.26	5.5	18.1	0.59	3.1	23.8
490	14.03	9.4	31.4	10.87	7.2	29.0	7.39	4.1	27.1	4.57	4.1	15.2	5.64	2.0	18.8	8.04	8.8	29.0	9.93	5.1	19.8	7.29	3.9	24.1	9.86	7.7	20.0	12.89	9.0	24.4	16.43	11.7	41.4	19.92	7.0	44.5
491	0.97	7.3	7.7	0.43	1.7	1.8	0.73	2.4	3.9	1.24	5.6	6.8	4.46	8.3	28.8	9.98	9.9	26.0	17.64	11.5	49.9	16.34	10.9	78.1	14.13	13.2	57.9	7.66	7.6	23.2	5.52	11.0	23.4	2.43	3.5	13.7
492	1.21	3.6	5.8	0.67	1.3	2.1	1.92	2.2	5.9	4.55	5.1	9.7	19.06	16.4	55.0	15.80	19.6	38.5	40.30	46.0	93.0	41.50	23.0	99.0	31.80	31.5	88.3	16.00	27.1	59.5	3.30	4.9	9.3	2.09	7.2	10.9
493	7.57	2.7	38.2	3.83	2.8	19.1	2.45	2.1	7.5	1.78	3.9	8.2	4.41	5.2	13.3	5.32	6.6	13.8	7.14	4.1	19.9	9.41	7.6	19.7	11.30	6.0	23.4	12.00	9.3	30.6	12.30	11.3	34.6	12.68	7.1	25.3
494	31.42		48.0	23.60		37.1	24.39		35.2	12.14		16.6	17.20		25.6	12.92		21.0	11.60		24.2	12.24		24.5	27.70		68.7	37.65		59.0	34.20		99.5	37.62		67.5
495	4.62	2.0	9.5	5.04	2.2	9.9	6.91	2.8	15.7	5.46	4.4	11.2	9.20	3.0	20.1	6.27	3.1	8.5	7.05	3.6	21.7	10.15	4.2	30.8	2.30	8.1	5.66	6.25	2.9	6.7	12.25	3.1	10.3			
496	3.10		7.7	3.47		7.6	5.21		9.2	8.95		20.1	7.35		15.8	13.56		26.8	9.27		31.4	7.24		19.0	8.57		22.6	4.61		12.9	3.36		8.5	3.26		8.6
497	2.44		6.4	2.64		6.6	4.44		7.8	6.26		12.8	5.57		11.3	9.26		24.7	7.56		24.7	5.70		13.7	7.34		17.7	5.25		10.1	3.01		6.9	2.15		4.7
498	2.20		5.9	2.65		6.6	4.49		7.1	4.92		9.4	4.92		9.4	7.30		15.6	5.30		12.6	3.74		16.2	5.76		14.4	3.72		14.0	2.22		10.0	2.12		6.2
499	2.59		6.3	2.65		6.4	3.45		7.1	3.69		8.2	4.69		16.3	4.07		14.3	5.26		16.1	7.02		23.5	8.49		16.3	6.66		15.4	4.22		10.6	2.48		9.1
500	1.48		3.7	1.00		4.2	2.19		7.9	2.87		5.4	3.74		7.9	3.06		8.5	3.79		6.2	4.34		11.9	5.54		10.6	3.82		8.4	3.25		9.2	2.30		7.4
501	0.08		0.4	0.16		0.4	0.28		0.8	1.12		2.9	1.25		3.2	1.47		1.9	2.18		1.9	3.45		13.9	2.36		4.4	1.61		4.9	0.50		2.4	0.19		0.6
502	0.79		2.2	0.54		2.4	1.05		6.8	2.57		6.1	3.20		5.8	4.26		11.5	5.57		17.9	8.16		15.9	4.26		17.3	1.57		3.5	1.70		3.0	0.74		2.1
503	0.12	0.9	1.2	0.19	0.5	1.1	0.25	0.7	1.1	0.64	1.3	2.2	1.53	2.4	5.3	3.36	4.6	8.1	8.40	9.9	19.6	6.47	5.7	14.6	2.65	4.1	11.2	0.61	1.5	1.8	0.28	0.9	0.9	0.07	0.5	0.8
504	2.02		7.8	2.28		6.1	3.20		6.3	3.71		9.4	3.55		7.2	7.27		19.4	6.02		13.0	5.07		13.5	4.45		10.8	3.22		12.0	2.07		5.9	1.39		6.0
505	10.98			1.39			3.27			5.43			12.45			16.26			13.83			14.80			12.52			5.25			1.14			0.94		4.9
	1.32		8.4	1.63		7.9	2.72		11.5	5.36		14.9	11.05		48.8	15.93		34.4	13.83		30.1	14.12		30.1	9.04		30.6	6.88		24.0	1.85		8.8	1.14		
506	0.90		4.3	0.13		0.4	0.28		1.3	1.76		7.8	8.35		11.8	12.61		16.6	11.11		23.4	10.96		19.7	13.49		18.2	11.07		23.7	3.67		11.2	3.18		6.8
507	0.79		5.2	1.91		6.6	2.80		10.4	5.30		23.1	17.35		36.4	30.24		62.7	31.48		53.5	29.54		49.6	26.35		43.8	12.42		25.9	3.48		11.9	0.65		6.9
508	1.45		23.0	1.12		8.0	0.65		6.1	2.42		13.3	15.92		40.6	18.91		41.5	15.32		31.2	15.06		34.3	18.07		32.3	11.41		20.3	9.12		25.5	6.17		17.6
509	0.15		7.8	0.10		2.3	0.36		3.7	2.94		10.5	20.61		45.8	37.65		64.0	45.10		68.3	43.75		72.2	29.05		63.3	8.49		21.0	1.72		10.7	0.17		3.8
510	0.18		5.5	0.26		4.4	0.26		4.0	3.71		9.4	3.55		7.2	7.27		19.4	6.02		13.0	5.07		13.5	4.45		10.8	3.22		12.0	2.07		5.9	1.39		6.0
511	0.11		1.5	0.18		3.6	0.51		4.9	2.01		15.8	13.70		62.1	49.42		97.4	53.68		97.8	42.47		95.2	34.58		54.2	11.98		27.4	4.98		35.2	0.57		7.4
512	0.05		0.9	0.09		0.6	0.19		1.6	1.12		4.4	5.78		11.1	5.52		13.6	3.29		9.2	4.68		10.5	5.74		14.1	6.72		14.2	1.68		9.1	0.38		1.9
513	0.52		2.3	0.90		6.2	2.46		8.6	6.46		13.4	9.31		24.1	12.53		34.2	12.23		39.0	10.66		19.8	7.44		18.2	2.81		13.3	0.54		5.0	0.22		1.9
514	0.43		2.2	0.91		3.5	2.21		6.1	4.08		11.8	10.10		25.9	13.64		24.4	16.22		55.9	13.24		30.6	12.37		32.8	6.05		20.3	2.24		10.2	0.23		1.9
515	0.67		4.3	2.31		18.0	10.57		54.9	31.28		93.3	50.85		141.8	103.6		200.1	107.4		266.1	81.47		149.3	49.41		125.1	16.75		58.1	2.27		13.4	0.26		3.2
516	0.42		2.2	0.68		4.8	1.68		6.4	2.16		8.4	3.56		9.0	4.91		11.1	4.18		9.4	3.50		9.0	3.10		8.1	2.45		7.6	1.34		5.2	0.19		1.6
517	0.61		3.7	0.63		2.8	0.41		2.8	0.31		2.0	1.61		9.6	7.73		28.9	11.30		23.4	16.91		26.2	2.59		14.0	0.25		2.8	0.11		1.9			
518	0.54		4.8	0.54		4.6	0.57		3.8	0.53		3.9	0.78		5.9	9.79		22.2	13.34		24.8	10.34		22.4	7.87		24.7	2.06		10.5	0.53		4.8	0.48		8.2
519	0.26		3.6	0.48		6.8	0.32		2.9	0.71		4.2	2.31		4.9	5.15		14.6	5.08		12.5	7.21		22.0	9.39		30.3	4.15		15.6	1.23		6.3	0.25		2.8
520	0.29		3.2	0.34		8.9	0.32		4.0	0.48		4.5	1.30		9.5	4.32		11.7	6.10		16.7	6.46		19.9	6.53		17.4	8.21		29.4	4.70		20.7	0.69		7.2
521	1.14		9.6	0.91		6.8	0.70		5.2	0.64		7.5	2.64		23.3	6.64		13.2	6.64		11.9	4.54		13.2	4.85		14.0	13.15		27.7	13.61		42.8	5.22		22.8
522	0.09		29.1	2.12		5.1	1.55		8.2	2.03		9.4	2.49		9.2	1.36		6.5	2.08		7.8	4.20		11.7	4.68		11.4	7.85		20.7	14.19		35.2	13.79		31.5
523	5.57		24.7	2.09		10.8	3.36		9.6	5.60		13.5	6.61		22.1	12.80		27.8	11.90		35.0	7.83		20.8	8.06		34.5	10.97		28.5	9.05		16.8	8.72		23.1
524	3.23		12.6	1.94		5.3	4.28		10.5	9.73		28.8	10.94		22.8	7.32		20.0	4.43		28.0	3.24		19.5	4.76		25.1	13.36		35.2	11.76		25.8	5.12		15.5
525	2.88		9.6	2.31		6.5	2.05		5.3	4.23		10.0	6.02		11.2	4.06		8.6	5.02		11.5	6.99		16.0	7.23		13.6	9.68		18.6	8.16		15.5	4.42		12.1
526	0.82		5.2	0.78		5.8	1.79		17.9	2.66		14.7	11.40		27.1	27.79		49.5	25.27		44.3	12.54		29.4	9.21		22.5	12.95		27.1	6.70		16.5	1.91		7.2
527	0.11		4.2	0.19		3.1	0.38		3.1	1.72		11.7	6.88		44.4	28.03		76.4	32.79		43.9	20.10		47.2	17.56		37.1	6.53		19.0	2.84		10.2	0.31		2.9
528	0.24		4.0	0.15		3.1	0.54		6.4	1.34		5.0	4.66		15.5	3.09		8.6	4.02		11.4	5.61		14.2	6.83		19.3	5.99		19.5	2.24		9.5	0.58		5.8
529	0.11		2.3	0.04		0.9	0.37		3.0	1.83		6.7	2.66		10.6	8.65		18.9	15.73		42.2	9.42		27.0	4.37		12.4	4.65		11.9	1.45		10.7	0.36		7.6
530	0.11		2.3	0.04		1.0	0.37		3.0	0.04		1.6	0.68		10.3	0.03		6.2	1.24		17.4	1.56		17.4	0.60		11.3	1.31		9.6	0.45		6.1	0.13		1.1
531	0.02		0.4	0.19		1.0	0.05		0.6	0.03		0.5	0.51		4.2	3.88		17.1	11.82		37.5	8.29		34.7	4.00		20.1	9.56		7.1	0.14		2.4	0.03		0.5
532	0.75		5.0	0.55		3.1	0.39		3.5	0.15		1.4	0.31		2.5	4.68		20.7	12.01		23.7	10.99		30.5	6.32		18.7	2.32		12.5	0.31		2.3	0.21		2.9
533	0.41		2.6	0.28		2.2	0.35		1.8	0.20		2.5	0.61		2.9	2.59		13.6	8.28		21.6	7.30		21.6	9.19		14.7	0.30		2.8	0.13		2.4	0.30		1.5
534	3.66		17.0	1.72		3.3	1.73		9.1	2.74		6.4	2.02		8.1	21.04		44.2	14.74		46.2	7.48		16.5	5.74		14.2	1.48		9.2	0.30		2.4	0.30		1.1
535	0.36		1.6	0.31		1.7	0.27		1.5	0.24		1.3	0.23		2.2	0.19		1.1	0.47		1.9	0.52		2.0	0.26		2.7	0.16		4.0	0.03		0.2	0.16		1.1
536	0.98		3.6	0.31		1.7	0.86		3.2	0.56		3.0	0.72		4.2	0.72		4.2	0.72		4.2	0.72		4.2	0.72		4.2	0.72		4.2	0.72		4.2	0.72		4.2	0.72		4.2
537	1.53		5.1	1.37		5.0	2.10		7.5	1.82		7.3	0.72		5.2	0.32		3.8	1.41		6.9	2.17		17.1	0.31		7.0	0.20		1.6	0.42		3.5	0.63		4.3
538	1.86		6.4	1.81		7.5	1.86		5.3	1.01		5.1	0.27		2.0	0.15		1.1	0.36		2.7	0.42		2.5	0.07		1.1	0.13		1.9	0.32		2.7	0.03		4.2
539	1.47		3.4	0.97		3.4	0.57		2.5	0.24		1.7	0.23		2.2	0.47		3.6	2.81		8.9	15.8		2.2	0.60		9.0	0.13		1.9	0.32		2.7	0.03		4.2
540	0.22		1.9	0.29		2.2	0.31		3.0	0.12		3.4	0.15		2.2	0.47		3.6	2.81		8.9	15.8		2.62		11.8	0.62		5.7	0.01		0.4	0.07		1.9	0.05		0.6
541	0.58		3.4	0.36		2.9	0.20		3.6	0.12		4.7	0.07		1.8	0.04		1.0	2.82		15.6	1.68		3.6	0.04		1.6	0.03		1.0	0.03		1.4	0.17		1.4
542	1.16		4.5	0.84		3.9	0.59		2.5	0.40		3.9	0.01		0.2	0.12		2.5	0.02		0.3	0.03		0.6	0.00		0.0	0.00		0.0	0.03		1.0	0.32		3.0	0.59		2.5
543	1.13		4.5	0.68		3.1	0.89		4.8	0.18		2.3	0.01		0.4	0.06		0.8	0.003		0.1	0.01		0.3	0.00		0.0	0.05		0.6	0.27		2.0	1.12		4.7
544	2.67		8.0	1.92		6.3	1.47		6.0	0.40		2.5	0.01		0.4	0.06		0.8	0.005		0.2	0.03		0.6	0.00		0.0	0.05		0.6	0.27		2.0	1.12		4.7
545	0.82		1.9	1.00		3.3	2.24		5.2	1.84		3.9	1.23		4.0	3.33		2.1	0.06		1.3	0.03		0.3	0.02		0.1	0.59		1.6	0.60		2.3	0.05		2.3	0.3
546	0.44		2.2	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0	0.47		2.0
547	1.17		4.6	1.22		7.9	1.34		5.1	0.93		3.2	0.23		1.2	0.08		0.5	0.04		0.3	0.03		1.1	0.01		1.2	0.08		0.5	0.74		4.8	1.23		4.5		
548	0.32		1.8	0.19		1.6	0.48		6.6	0.16		2.1	0.12		1.4	0.06		1.3	0.03		0.6	0.12		2.0	0.15		1.4	0.09		2.2	0.06		1.3	0.11		1.1
549	0.00	Tr.	Tr.	0.00	Tr.	Tr.	0.00	Tr.	Tr.	0.00	Tr.	Tr.	0.12	0.8	0.9	0.31	1.3	1.4	1.76	3.5	5.9	2.60	2.2	7.4	0.79	2.0	1.9	0.30	0.5	2.0	0.00	0.0	0.0	0.00	0.0	0.0
550	0.04	0.7	0.7	0.20	1.0	1.0	0.94	1.7	2.4	2.52	2.1	5.4	4.90	3.9	10.2	6.80	2.2	9.8	6.12	4.9	16.8	7.60	5.4	15.1	7.76	2.2	23.9	7.54	4.5	13.1	6.63	1.0	1.6	0.00	0.0	0.0
551	1.06	1.9	2.2	1.77	1.3	5.2	2.76	1.6	8.7	4.50	2.1	10.3	9.25	1.9	13.7	10.12	2.0	13.7	10.83	1.6	16.8	11.57	3.5	15.3	11.05	2.0	17.0	7.75	2.2	14.4	3.59	2.9	7.7	1.42	1.3	2.5
552	2.59	1.2	2.9	1.80	1.5	6.1	2.76	1.6	7.9	3.43	1.7	5.5	2.35	2.1	10.6	4.75	1.9	11.0	6.96	1.6	15.0	6.09	3.0	16.5	7.85	2.0	22.1	6.79	1.6	5.8	0.95	1.7	3.3	0.54	1.1	3.3
553	2.61		8.0	3.60		8.2	5.81		9.9	9.75		15.8	8.51		15.8	5.11		10.3	2.94		7.7	3.06		6.7	2.07		5.6	3.47		6.5	5.01		12.0	5.08		12.5
554	3.27		10.2	2.13		6.0	4.82		10.5	11.84		23.8	7.38		14.8	1.10		4.6	1.66		2.4	1.12		4.2	1.15		2.6	1.24		4.7	2.86		10.6	3.74		9.7	
555	2.83		6.8	2.20		10.7	6.06		10.6	14.07		33.3	10.71		21.2	1.93		8.1	2.36		5.1	1.65		4.5	2.11		6.8	2.03		6.8	7.48		17.2	5.36		17.8	
556	16.90		31.4	12.81		34.1	12.66		32.4	11.84		23.8	7.38		14.8	1.10		4.6	1.66		2.4	1.12		4.2	1.15		2.6	1.24		4.7	2.86		10.6	3.74		9.7	4.5
557	12.01		24.3	11.48		26.4	7.41		15.2	2.02		5.2	0.57		1.6	0.27		1.0	0.23		0.7	0.31		1.3	0.57		2.5	2.50		11.1	5.07		15.4	11.25		21.6
558	7.47		15.7	7.40		18.8	4.50		11.4	0.99		2.9	0.54		4.9	0.03		0.7	0.03		0.4	0.08		0.3	0.26		3.1	1.14		4.2	3.70		8.4	5.78		12.7
559	5.93		13.7	6.04		11.8	3.55		9.2	0.65		3.0	0.28		1.2	0.03		0.5	0.04		0.9	0.03		0.5	0.12		1.3	0.91		3.6	2.31		7.7	5.18		13.4
560	16.48			14.71			13.69			4.02			2.60			0.62			0.89			1.72			1.64			4.94			8.46			11.62		
561	6.11		20.0	5.22		16.0	4.44		10.9	1.74		5.2	0.76		4.1	0.18		0.8	0.33		3.4	0.51		4.8	0.96		2.8	2.56		8.8	4.96		19.2	5.43		10.6
562	4.07			3.44			3.05			1.44			0.96			0.41			0.58			0.85			0.38			1.98			2.46			3.2		
563	2.91		12.2	3.26		8.4	3.25		10.1	1.94		4.2	0.96		4.1	0.58		3.4	0.41		2.7	0.72		3.1	0.85		4.4	1.41		4.3	1.82		9.3	2.46		9.2
564	4.44			3.67			3.44			1.94			0.96			0.41			0.58			0.85			0.38			1.98			2.46			3.2		
565	4.57		9.5	4.91		11.1	5.37		21.1	3.39		15.4	1.90		5.8	1.16		11.1	1.24		15.2	1.74		7.9	3.18		13.8	5.14		26.7	4.97		16.6	5.10		13.8	
566	1.21		4.9	1.33		5.5	1.79		5.3	1.97		5.5	2.18		8.0	1.68		4.8	1.89		6.3	2.09		5.0	2.24		7.7	2.14		6.9	2.09		9.3	1.70		6.3
567	0.70		5.1	0.58		3.1	0.91		4.4	1.82		5.9	2.54		13.3	2.15		8.6	2.39		8.6	2.39		8.6	2.37		6.0	1.64		4.7	1.66		7.2	0.42		3.2
568	0.35			0.33			0.57			0.92			0.64			0.55			0.23			0.38			0.30			0.40			0.56			0.47		
569	0.19			0.33			0.57			0.74			1.33			1.65			1.32			1.32			0.81			0.60			0.28	

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record. For day or less.	Years of Record. For month and year.	Years of Record. Last year included.	Maximum Observed Precipitation. Number of Minutes. 5.	15.	60.	Day or 24 hours.	Annual Precipitation. Month.	Annual Precipitation. Minimum.	Annual Precipitation. Mean.	Annual Precipitation. Maximum.	Observed Precipitation, in Percentages of Mean Annual Rainfall. Mean daily precipitation on days of rainfall.	Maximum During short Periods. Number of Minutes. 5.	15.	60.	Mean for month of maximum precipitation	Maximum. In 24 hours.	Maximum. In one month.	Annual. Minimum.	Annual. Mean.	Annual. Maximum.	Main references.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
592	Elisabethville, S. E. Congo			14	1925							45.2									100		15	
593	Gazi, N. E. Congo			12	1922							72.7									100		15	
594	Kala, N. Congo			16	1922							70.9									100		15	
595	Boma, W. Congo			18	1922							35.7												
596	St. Helena Island	1 983		29	1920					11.0	51.0	40.1	49.8					12.3		27.4	77	100	124	4
597	Calabar, Nigeria	42		24	1920					41.2	90.0	123.2	167.4					14.8		33.5	73	100	136	4
598	Lagos, Nigeria	7		29	1920					31.0	40.5	71.4	115.5					26.1		43.4	57	100	162	4
599	Akkra, Gold Coast	61		27	1920					21.1	13.1	27.2	44.2					25.8		77.9	48	100	163	4
600	Gambaga, W. Africa	350		20	1920					18.2	30.7	45.8	71.7					24.1		39.9	67	100	157	4
601	Timbuktu, W. Africa			15	1914							7.7									100			
602	Freetown, Sierra Leone	189		46	1920					61.1	102.5	157.2	206.6					23.3		38.9	65	100	132	4
603	Bathurst, Gambia	7		27	1920					36.6	24.6	47.6	80.8					41.1		77.0	52	100	170	4
604	La Laguna, Canary Islands	10		26	1920					20.6	10.3	23.4	41.7					16.3		87.8	44	100	178	4
605	Funchal, Madeira Islands	82		41	1920					19.0	10.7	27.2	55.8					14.6		69.6	39	100	204	4
606	Cape Spartel, Tangiers	193		28	1920					14.1	21.9	30.4	46.8					18.2		46.4	72	100	154	4
607	Aqiers, Algeria	{1 129 / 13		27 / 54	1920 / 1896					11.0	12.7	27.4 / 28.9	42.7					14.6		40.2	46	100	156	4 18; 15
608	Tunis, Tunis	69		31	1920					7.1	11.0	14.5	22.8					11.3		43.0	67	100	188	4
609	Alexandria, Egypt	112	37	39	1922				11.9	15.4	3.2	8.0	23.0					22.6	137.5	192.7	40	100	288	4; 19
610	Cairo (Abbassia), Egypt	112	34	39	1922				1.4	2.0	0.2	1.3	5.4					21.5	154.0	154.0	0	100		4; 19
611	Aswan, Egypt	331		20	1920							0.0									100		19	
612	Suez, Egypt	82	34	35	1920				1.2	2.5		1.8	7.5					12.0	120.0	250.0		100	360	19; 18
613	Ismalli, Suez Canal	39	18	35	1920				2.1	2.7		1.8	4.1	2.5				23.8	116.4	149.5		100	227	19
614	Gaza, Palestine	66		19	1905							16.5									100		15	
615	Jerusalem, Palestine	1 968		153 / 144	1912 / 1895					16.4	13.4	25.0 / 25.5	45.0	1.7				21.8		65.0	53	100	178	15
616	Tiberias, Egypt	—653		16								16.1									100		16	
617	Le Kexy, Egypt	3 330		11	1914							59.6									100		15	
618	Beirut, Syria	111		45	1920					15.7	23.2	35.5	51.1					16.2		43.2	65	100	144	4
619	Nicosia, Cyprus	525	14	14	1920				8.7			15.1						12.1	57.5			100		19
620	Kyrenia, Cyprus	40	14	14	1920				5.2			21.1						25.2	24.6			100		19
621	Candia, Crete	128	13	13	1920				4.4			21.1						20.1	20.8			100		19
622	Athens, Greece	356		26	1920					8.1	4.6	15.5	22.4					16.8		52.0	30	100	144	4
623	Smyrna, Turkey			29	1882							25.6		1.4							100		20	
624	Constantinople, Turkey	246		48	1895				5.3			28.6						5.7	21.7		100		15	
625	Batak, S. W. Bulgaria	3 180		20	1918							13.9	20.6	36.5							52	100	137	14
626	Sofia, W. Bulgaria	1 803		20	1918							15.6	24.3	33.3							64	100	137	14
627	Petrohan, N. W. Bulgaria	4 588		15	1913							31.4	45.3	61.2							70	100	132	14
628	Ferdinand, N. W. Bulgaria	525		20	1918							8.1	21.4	31.8							38	100	148	14
629	Varna, N. E. Bulgaria	115	15	19	1918				3.9			14.8	20.3	26.2				8.3	19.2		73	100	128	14; 22
630	Sulina, E. Roumania	7		58	1925					10.3	5.2	15.4	24.4					8.5		67.0	34	100	158	4
631	Braila, E. Roumania	52	20	32	1920				3.6			13.6	22.6					7.5	21.2		53	100	142	22
632	Bucharest, S. Roumania	269		61	1924					11.7	12.5	23.1	34.0					15.0		50.8	58	100	147	4
633	Belgrade, Jugo-Slavia	452		35	1924					8.0	12.8	24.4	32.8					9.7		52.6	52	100	138	4
634	Lesina, S. W. Jugo-Slavia	66		61	1918					18.1	8.7	17.1	31.2					14.0		40.8	53	100	146	4
635	Obir, Austria	6 710		45	1923					21.1	36.1	60.2	91.5					5.7		35.0	60	100	153	4
636	Vienna, Austria	663		70	1920					9.0	16.5	25.5	35.6					10.7		35.2	65	100	140	4
637	Sonnblick, Austria	10 020		33	1923					15.3	41.3	69.0	79.5					7.9		23.3	63	100	140	4
637-A	Klagenfurt, Austria	1 470		88	1900					12.7	16.1	28.5	37.3					7.8		32.9	42	100	150	15; 33
638	Santis, Switzerland	8 200		38	1920					27.5	53.3	100.3	154.2					8.6		27.4	53	100	154	4
639	St. Gallen, Switzerland	2 230		50	1920							53.7									100		23	
640	Zurich, Switzerland	1 566		57	1920					16.9	29.0	43.6	76.3					12.0		28.6	66	100	180	
641	Chaist Capt, Switzerland	4 423		50	1900							81.7									100		9	
642	Interlaken, Switzerland	1 870		50	1919							46.9									100		23	
643	Geneva, Switzerland	1 329	30	50	1920				4.9			35.3						11.6	13.9			100		23; 9
644	Gothard, Switzerland	6 890		20	1920							60.1									100		23	
645	Locarno, Switzerland	670		70	1905							76.5									100		23	
646	Domodossola, Italy	984		26	1905					25.3	32.8	55.3	73.6					13.1		45.6	59	100	133	23
647	Aosta, N. W. Italy	1 912		60	1910							22.4									100		23	
648	Valdobbia, N. W. Italy	3 250		50	1920					18.1	36.8	66.0	76.1					7.8		38.7	75	100	164	11
649	Milan, N. Italy	482		101	1924					14.8	24.9	39.4	61.8					12.0		37.6	62	100	157	4
650	Pavia, N. Italy	269		92	1908					16.3	19.0	33.3	51.3					10.7		33.5	70	100	164	23
651	Venice, N. E. Italy	154		26	1905					8.8	19.1	26.2	38.3					6.1		32.6	73	100	146	11
651-A	Padua, N. E. Italy	103	17	202	1926				3.5	12.3	18.9	33.8	51.6	1.0				9.8	10.7	36.4	68	100	162	15; 23
652	Camaldoli, N. Central Italy	3 680		51	1913					19.7	44.5	71.1	111.1					11.7		42.4	62	100	171	4
653	Florence, N. Central Italy	330		24	1905					10.7	15.9	33.0	44.8					11.8		32.4	48	100	136	11
654	Rome, Italy	208		143	1924					13.9	12.6	32.7	57.9					15.1		38.6	52	100	162	4
655	Naples, Italy	532		24	1905					10.3	17.4	33.6	44.6					12.1		30.6	52	100	122	11
656	Oppido Mamertina, S. Italy	1 223		44	1925					22.1	21.6	51.4	91.7					12.3		49.1	64	100	139	11
657	Catania, Sicily	213		42	1923					19.8	12.9	24.6	42.7					18.4		80.3	51	100	177	4
658	Ramari, Sardinia	79		27	1920					19.6	17.0	32.6	58.2					15.6		44.9	72	100	162	4
659	Genoa, Italy	259		26	1905					15.7	34.3	52.9	75.3					14.1		35.4	65	100	142	11
660	Nice, France	1 140	30	30	1910					5.7		22.3						18.4	17.6			100		9
661	Marseilles, France	246	30	50	1920					9.7	12.4	12.3	43.5					17.0	38.6	54.9	54	100	190	4; 9
662	Lyon, France	981	30	80	1920					4.2	11.3	19.3	29.1	45.2				12.7	14.8	38.8	66	100	155	4; 9
663	Besancon, France	1 020	30	30	1910					2.6			28.8					9.2	6.6			100		9
664	Nancy, France	724	30	30	1910					2.0			28.2					10.3	9.0			100		9
665	Paris, France	164	30	50	1923					2.0	6.2	10.9	22.7	29.5				10.1	10.2	27.5	48	100	131	4; 9
665-A	Paris, France	164	30	214	1926					2.0	6.9	9.0	19.9	29.8				9.5	10.0	34.6	45	100	150	15; 32
666	Clermont, France	1 308	30	30	1910					2.5			25.9					10.1	10.8			100		9
667	Puy de Dome, France	4 812	30	30	1910					2.5			35.7					9.9	9.8			100		9
668	Perpignan, S. E. France	160	30	50	1910					5.8	15.9	11.5	21.6	39.2				11.1	20.6	73.5	53	100	191	24; 9
669	La Preute, S. E. France	3 080		17	1925						15.7	25.0	34.3	51.6				5.1		35.5	56	100	140	24
670	Puymorens, S. E. France	6 320		28	1900						14.0	20.1	31.4	47.4				10.5		44.5	64	100	151	24
671	Huesca, N. E. Spain	1 653	20	29	1900					3.4	11.4	11.2	22.2	33.7				11.5	15.3	51.8	50	100	152	26
672	Haragossa, N. E. Spain	672	19	40	1899					2.6	6.2	5.7	21.1	38.5				13.5	21.7	64.1	58	100	184	26
673	Barcelona, Spain	70		30	1910						13.3	10.7	21.1	38.5				15.6		63.0	51	100	152	4
674	Palma, Spain			56	1921						9.3	9.4	18.9	30.6				11.5		48.2	50	100	138	4
675	Valencia, Spain	59		30	1890						23.6	7.5	18.9	30.7				15.6		72.0	38	100	245	30
676	Madrid, Spain	2 149		60	1919						7.3	10.2	16.8	27.5				9.9		44.0	61	100	166	4
677	Gibraltar, Gibraltar	53		47	1920						24.8	24.6	33.7	54.7				10.2		44.0	54	100	166	30
678	Seville, S. W. Spain	98		29	1890						12.3	6.3	16.6	24.7				11.7		66.0	34	100	186	4
679	Lisbon, Portugal	312		57	1920					15.1	17.3	29.3	55.9					13.9		51.4	59	100	190	4
680	Serra de Estrella, Portugal	4 756	19	19	1900				11.5	48.6	21.1	51.0	75.8	0.8				12.8	10.1	43.0	53	100	163	26
681	Oporto, Portugal	328	23	38	1900				6.5	20.4	23.8	51.4	89.0	0.8				10.8	12.7	39.8	46	100	173	26
682	Valladolid, Spain	2 345	20	40	1900					2.2	8.2	5.6	12.5	17.4	1.0			11.0	17.6	65.6	45	100	139	26
683	Oviedo, N. W. Spain	800	50	50	1900					3.2	16.4	25.2	36.0	49.8	1.4			10.0	8.9	31.5	70	100	138	26
684	Bilbao, N. Spain	56	20	46	1900					4.6	18.6	30.2	47.2	64.4				11.1	10.2	41.5	64	100	137	26
685	Pic du Midi, N. Spain	9 380		30	1910					4.9			65.5					7.3	7.5			100		9
686	Bayonne, S. W. France		756	30	1890							45.3									100		30	
687	Toulouse, S. W. France		635	30	1910					3.5		30.3						12.3	13.8			100		9
688	Bordeaux, S. W. France		243	30	1910					2.5		30.3						9.2	8.2			100		9
689	Nantes, W. France		121	30	1920					2.2	10.0	22.1	30.9	41.1				12.8	9.4	32.3	72	100	133	4; 9
690	Brest, W. France		200	30	1910					2.8			32.3					9.4	8.7			100		9
691	Jersey Island	25	39	51	1921		1.59		2.4	10.8	15.9	33.5	45.6					12.2	7.2	32.2	47	100	136	25; 8
692	Arras, N. France		207	30	1910					2.1			26.5					9.6	7.9			100		9
693	Calais, N. France		23		38	1910							29.1									100		9
694	Dungeness, S. E. England		21	30	50	1921		1.60	2.8	3.4	10.4	12.8	24.4	46.6				11.0	9.8	42.6	56	100	191	25
695	Greenwich, England		150	81	81	1921			2.6	2.7	7.6	12.5	24.3	35.6				11.0	15.2	31.3	61	100	160	4; 25
696	London, Kew Obs., England		18	61	51	1921	0.7	1.4	3.28	2.4	7.2	12.2	23.8	38.2		2.9	5.9	13.7	9.0	10.1	30.1	51	100	169	25
697	Southampton, England		64	43	51	1921				2.9	11.1	16.1	30.9	43.2				12.8	9.4	36.0	52	100	140	25
698	Falmouth, S. W. England		167	51	51	1921				2.8	11.0	28.9	43.6	60.6				14.3	6.4	25.2	66	100	139	25
699	Scilly Island, England		133	41	51	1921				3.1	8.5	21.6	31.9	45.3				12.7	9.7	27.6	68	100	142	25
700	Cardiff, S. Wales		202		35	1915		1.02					41.4									100		25
701	Lluchfa, N. Wales		2 500		35	1917							101.0									100		13
702	Holyhead, England		13	46	51	1921				3.4	10.4	14.0	34.9	56.2				11.4	9.7	29.8	69	100	161	25

(Continued.)

Station No.	JANUARY.			FEBRUARY.			MARCH.			APRIL.			MAY.			JUNE.			JULY.			AUGUST.			SEPTEMBER.			OCTOBER.			NOVEMBER.			DECEMBER.		
	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
582	9.01	.....	.....	9.54	.....	.....	8.74	.....	.....	1.81	.....	.....	0.23	.....	.....	0.00	.....	.....	Tr.	.....	.....	0.01	.....	.....	0.14	.....	.....	1.16	.....	.....	4.42	.....	.....	10.11	.....	.....
581	2.65	.....	.....	3.85	.....	.....	5.24	.....	.....	5.23	.....	.....	7.24	.....	.....	5.46	.....	.....	5.73	.....	.....	5.30	.....	.....	8.52	.....	.....	7.95	.....	.....	8.46	.....	.....	4.20	.....	.....
584	3.75	.....	.....	4.94	.....	.....	4.92	.....	.....	5.20	.....	.....	7.23	.....	.....	4.22	.....	.....	4.02	.....	.....	5.96	.....	.....	7.26	.....	.....	8.76	.....	.....	6.55	.....	.....	6.73	.....	.....
585	4.33	.....	.....	5.11	.....	.....	5.57	.....	.....	7.37	.....	.....	2.18	.....	.....	0.01	.....	.....	0.01	.....	.....	0.15	.....	.....	0.43	.....	.....	2.02	.....	.....	5.46	.....	.....	5.07	.....	.....
586	2.97	.....	5.7	3.80	.....	8.9	4.94	.....	11.0	3.87	.....	10.1	4.15	.....	6.7	4.07	.....	7.9	4.03	.....	8.3	3.72	.....	7.7	3.02	.....	5.5	1.91	.....	4.3	1.68	.....	3.9	1.96	.....	4.2
587	1.76	10.6	22.2	2.22	.....	8.5	6.22	.....	16.4	8.57	.....	14.7	12.61	.....	23.7	16.24	.....	32.6	19.47	.....	34.3	18.27	.....	41.2	16.61	.....	28.8	11.96	.....	20.3	7.59	.....	16.3	1.96	.....	7.2
588	1.07	8.1	20.7	2.07	.....	12.5	3.74	.....	11.3	5.75	.....	12.5	10.47	.....	21.1	18.65	.....	31.0	10.68	.....	29.9	2.80	.....	22.8	6.26	.....	15.9	7.76	.....	16.5	2.53	.....	7.0	0.80	.....	4.4
589	0.65	.....	3.5	1.00	.....	6.3	1.63	.....	8.8	3.70	.....	8.8	5.65	.....	13.6	7.00	.....	21.1	1.70	.....	8.9	0.61	.....	4.1	0.98	.....	5.2	1.94	.....	9.2	1.49	.....	3.9	0.69	.....	3.2
590	0.06	.....	0.9	0.05	.....	0.7	0.68	.....	2.0	2.55	.....	5.3	5.02	.....	10.5	5.68	.....	16.6	7.44	.....	12.7	10.97	.....	18.2	9.46	.....	14.6	2.92	.....	6.2	0.54	.....	1.9	0.20	.....	1.5
591	0.00	.....	.....	0.01	.....	.....	0.05	.....	.....	Tr.	.....	.....	0.24	.....	.....	0.80	.....	.....	2.59	.....	.....	2.61	.....	.....	0.86	.....	.....	0.15	.....	.....	Tr.	.....	.....	0.00	.....	.....
592	0.41	4.1	0.20	4.2	1.16	5.8	5.06	.....	13.2	11.47	.....	28.5	20.04	.....	46.0	35.58	.....	57.2	26.57	.....	61.1	28.48	.....	48.0	12.62	.....	32.9	5.12	.....	11.8	1.42	.....	7.2			
593	0.005	.....	0.1	0.013	.....	0.2	0.002	.....	0.1	0.014	.....	0.4	0.16	.....	2.2	2.95	.....	10.0	10.81	.....	27.4	19.60	.....	36.6	10.05	.....	19.8	8.69	.....	10.6	0.20	.....	2.1	0.08	.....	2.7
594	4.10	.....	9.9	3.45	.....	18.3	2.91	.....	9.1	1.76	.....	6.9	0.56	.....	2.2	0.20	.....	1.5	0.16	.....	1.1	0.09	.....	0.7	0.50	.....	1.7	2.38	.....	5.3	3.74	.....	7.4	3.83	.....	20.6
595	3.24	.....	9.3	3.59	.....	17.5	3.41	.....	12.8	1.94	.....	9.7	1.09	.....	4.9	0.37	.....	2.5	0.13	.....	1.4	0.13	.....	1.2	1.19	.....	6.2	3.97	.....	19.0	4.66	.....	18.9	3.23	.....	12.4
596	3.97	.....	11.8	3.36	.....	8.9	4.10	.....	9.1	2.17	.....	6.4	1.70	.....	5.2	0.52	.....	1.9	0.07	.....	1.0	0.07	.....	0.9	1.06	.....	5.2	3.04	.....	8.7	5.52	.....	14.1	4.78	.....	11.2
597	{4.02	.....	11.0	2.62	.....	4.9	3.31	.....	8.9	2.09	.....	5.0	1.73	.....	6.7	0.74	.....	2.5	0.13	.....	1.6	0.12	.....	0.9	1.17	.....	4.1	3.41	.....	8.3	4.13	.....	10.6	4.00	.....	7.7
	{3.82	.....		3.31	.....		3.12	.....		2.01	.....		1.30	.....		0.35	.....		0.09	.....		0.08	.....		1.02	.....		2.88	.....		2.92	.....		4.60	.....	
598	2.07	.....	6.3	2.01	.....	5.3	1.87	.....	7.1	1.48	.....	4.7	0.91	.....	2.8	0.50	.....	4.5	0.11	.....	0.7	0.21	.....	1.3	0.88	.....	4.3	1.86	.....	9.3	2.13	.....	8.7	2.38	.....	6.4
599	2.13	1.8	5.0	1.91	1.3	6.0	0.55	0.6	2.1	0.12	0.4	2.0	0.04	0.4	2.0	0.04	0.4	2.0	0.00	Tr.	0.0	Tr.	0.1	0.1	0.04	0.6	0.6	0.24	1.1	3.1	1.38	2.8	4.0	2.66	11.0	25.4
610	0.38	1.4	1.7	0.25	0.4	1.2	0.15	0.6	0.8	0.12	0.5	1.5	0.04	0.3	0.5	0.01	0.05	0.3	0.00	0.00	0.0	0.00	0.00	0.0	0.00	0.00	0.0	0.00	0.00	0.0	0.12	0.6	1.1	0.28	0.6	2.0
611	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....	0.00	.....	.....
612	0.24	0.8	2.1	0.12	0.5	0.8	0.12	0.7	0.9	0.12	0.4	1.2	0.12	1.3	2.5	Tr.	0.2	0.2	0.00	0.0	0.0	0.00	0.0	0.0	Tr.	Tr.	0.1	0.04	0.4	0.5	0.12	0.4	0.4	0.16	1.1	1.5
613	0.43	2.1	2.7	0.24	0.3	1.0	0.24	0.4	0.9	0.20	1.5	1.8	0.12	0.4	1.6	0.00	0.0	0.1	0.00	0.0	0.0	Tr.	Tr.	Tr.	Tr.	0.1	0.2	0.08	0.3	1.7	0.16	1.1	1.6	0.32	0.3	1.4
614	4.34	.....	.....	1.97	.....	.....	1.46	.....	.....	0.28	.....	.....	0.31	.....	.....	0.04	.....	.....	0.00	.....	.....	0.00	.....	.....	0.04	.....	.....	0.83	.....	.....	3.15	.....	.....	4.13	.....	.....
615	{6.38	.....	14.4	5.70	.....	12.2	2.56	.....	13.3	1.69	.....	6.5	0.27	.....	1.2	0.01	.....	.....	0.00	.....	.....	0.00	.....	.....	0.04	.....	0.8	0.41	.....	3.0	2.29	.....	8.0	5.50	.....	16.4
616	{6.30	.....		5.70	.....		2.57	.....		1.73	.....		0.28	.....		0.00	.....		0.00	.....		0.00	.....		0.00	.....		0.04	.....		0.55	.....		2.56	.....		4.33	.....	
617	5.52	.....		2.39	.....		2.30	.....		0.90	.....		0.15	.....		0.00	.....		0.00	.....		0.00	.....		0.00	.....		0.30	.....		2.45	.....		7.42	.....		10.54	.....	
618	12.81	.....		10.86	.....		9.19	.....		4.02	.....		1.91	.....		0.11	.....		0.01	.....		0.00	.....		0.00	.....		0.00	.....		0.00	.....		0.00	.....		0.00	.....	
619	7.33	.....	14.9	5.75	.....	15.7	3.92	.....	9.5	2.23	.....	9.4	0.84	.....	9.3	0.14	.....	2.7	0.02	.....	0.5	0.03	.....	0.3	0.30	.....	2.4	2.10	.....	8.1	5.25	.....	15.3	7.54	.....	13.4
620	2.59	1.8	.....	1.98	1.3	.....	1.75	1.9	.....	0.92	2.6	.....	1.09	3.2	.....	0.22	1.8	.....	0.01	0.1	.....	0.00	0.0	.....	0.35	1.8	.....	0.88	1.0	.....	1.83	3.7	.....	3.40	4.5	.....
621	4.36	2.8	.....	3.50	1.9	.....	2.29	2.3	.....	0.77	1.5	.....	0.17	0.9	.....	0.02	0.1	.....	0.00	0.0	.....	0.00	0.0	.....	0.28	1.2	.....	2.39	2.7	.....	5.23	6.7	.....	5.23	6.7	.....
622	3.57	1.7	.....	3.54	2.2	.....	1.75	1.3	.....	1.01	0.9	.....	0.30	1.6	.....	0.09	0.3	.....	0.04	0.8	.....	0.14	1.3	.....	0.64	2.9	.....	1.81	2.3	.....	3.43	2.6	.....	4.25	4.4	.....
623	2.06	.....	5.5	1.66	.....	4.0	1.10	.....	3.3	0.86	.....	2.7	0.83	.....	3.0	0.69	.....	4.7	0.28	.....	2.0	0.48	.....	2.9	0.60	.....	2.3	1.57	.....	4.6	2.61	.....	8.1	2.03	.....	5.8
624																						0.12	.....	.....									4.40	.....	.....
625	3.42	1.3	.....	2.72	1.4	.....	2.44	1.8	.....	1.65	1.2	.....	1.18	1.4	.....	1.34	1.6	.....	1.06	1.9	.....	1.65	6.3	.....	2.05	2.8	.....	2.62	2.4	.....	4.02	2.2	.....	4.80	2.6	.....
626	1.89	.....	.....	1.70	.....	.....	2.01	.....	.....	2.33	.....	.....	2.92	.....	.....	8.58	.....	.....	3.00	.....	.....	1.97	.....	.....	1.75	.....	.....	1.74	.....	.....	2.05	.....	.....	1.70	.....	.....
627	1.25	.....	.....	1.18	.....	.....	1.54	.....	.....	1.97	.....	.....	2.66	.....	.....	3.08	.....	.....	2.56	.....	.....	2.05	.....	.....	2.43	.....	.....	2.13	.....	.....	2.79	.....	.....	2.46	.....	.....
628	2.27	.....	.....	3.06	.....	.....	2.64	.....	.....	3.97	.....	.....	4.64	.....	.....	5.56	.....	.....	2.27	.....	.....	1.27	.....	.....	4.16	.....	.....	3.74	.....	.....	3.16	.....	.....	2.66	.....	.....
629	2.22	.....	.....	1.98	.....	.....	1.38	.....	.....	1.46	.....	.....	2.33	.....	.....	2.76	.....	.....	2.29	.....	.....	1.86	.....	.....	1.89	.....	.....	2.21	.....	.....	1.68	.....	.....	2.06	.....	.....
630	1.30	1.1	.....	1.14	1.7	.....	1.14	0.9	.....	1.34	1.3	.....	1.58	2.1	.....	2.72	1.6	.....	1.70	3.9	.....	1.42	3.5	.....	2.01	2.2	.....	2.01	2.2	.....	2.39	1.3	.....	1.83	1.5	.....
631	0.99	.....	3.3	0.84	.....	2.1	1.08	.....	4.1	1.12	.....	4.6	1.24	.....	4.7	1.90	.....	6.7	1.42	.....	5.1	1.28	.....	10.3	1.38	.....	5.0	1.52	.....	4.7	1.49	.....	5.0	1.08	.....	3.7
632	{1.35	2.2		1.19	.....	3.5	1.02	.....	4.2	2.39	.....	5.2	2.75	.....	7.6	3.00	.....	5.8	2.59	.....		1.78	1.9	.....	1.33	3.8	.....	1.41	1.1	.....	1.19	2.8	.....	1.20	1.0	.....
633	{1.26	2.2	5.0	0.96	0.9	3.8	0.83	1.0	3.4	1.44	1.4	4.6	1.79	1.1	5.2	2.80	3.4	11.7	2.68	7.2	6.3	1.44	2.1	7.0	1.06	6.7	5.5	1.00	4.1	6.3	1.46	1.3	6.3	1.46	1.3	5.3
634	1.24		4.5	1.09		5.8	1.65		4.7	1.73		5.2	2.47		6.8	3.46		11.7	2.68		7.2	2.00		7.0	1.56		5.5	1.60		6.3	1.64		6.3	1.64		5.3
635	1.31		3.9	1.19		3.5	1.62		4.2	2.39		5.2	2.75		7.6	3.00		5.8	2.59			1.78			1.74			2.05			1.70					
636	1.24		3.9	2.51		9.3	2.87		7.4	3.19		6.6	3.46		9.1	3.74		6.4	3.51		3.9	1.53		13.7	2.50		12.0	2.97		13.9	4.36		18.9	4.14		11.1
637	3.07		7.9	3.43		21.1	4.61		14.1	5.24		12.1	5.43		16.4	6.61		12.7	6.38		13.7	5.47		16.9	5.91		13.0	4.06		11.4	2.39		9.0			
638	3.46		5.0	1.30		5.2	1.81		4.8	2.05		9.3	2.80		7.3	4.72		9.0	3.11		6.1	2.72		7.2	1.67		4.8	1.85		5.2	1.47		11.0	2.35		12.5			
639	4.90		8.7	4.80		15.9	5.30		13.7	6.58		11.9	6.16		13.4	6.51		10.1	6.45		9.3	3.62		9.7	4.06		12.0	2.99		12.7	1.98		7.7			
639-A	1.50		5.5	1.34		5.9	1.97		5.9	2.56		6.6	3.74		10.2	4.52		10.9	4.72		11.3	4.92		9.3												
638	7.08		22.6	6.25		21.0	7.07		27.0	8.58		18.1	7.73		17.9	10.85		20.8	12.03		23.7	10.87		24.4	8.61		18.1	6.88		16.3	5.85		18.7	8.07		27.5
639	2.48			2.48			3.34			5.68			6.36			6.05			5.07			5.07			4.05			2.60			2.99					
640	2.46		6.8	2.15		7.5	2.90		11.3	5.58		8.1	4.20		10.2	5.24		16.9	5.11		10.9	5.07		9.6	4.04		9.7	3.56		8.5	2.67		7.7	3.01		7.1
641	5.01			5.32			6.85			6.30			7.28			8.24			6.98			6.42			4.66			3.58			2.72			3.66		
642	2.61			2.56			3.71			2.62			2.89			2.82			2.88			2.88			3.19	1.9		2.84			2.72			2.72	1.2	
643	1.69	1.8		1.97	2.0		2.52	1.8		2.73	1.8		2.87	1.3		3.27	2.0		3.19	1.8		3.77	2.5					4.20			5.39			3.14		
644	5.08			4.72			6.77			6.77			6.69			7.42			7.42			7.42						5.39			5.71			5.27		
645	2.84			2.78			5.17			9.35			8.55			7.17			7.37			9.21			8.59			9.60								
646	3.14		9.1	2.82		10.6	3.85		13.4	5.40		14.0	6.60		14.9	4.76		12.2	2.61		7.8	4.55		12.4	5.29		18.6	7.21		25.3	4.75		17.1	3.20		8.5
647	1.93			1.89			1.77			2.61			2.48			2.50			2.80			2.12			2.57			2.64			3.72			3.95		
648	4.00		14.2	3.61		16.6	5.60		16.9	5.60		10.4	5.02		10.0	3.14		5.0	2.80		5.1	2.12		5.5	2.57		9.6	2.64		18.1	3.72		11.2	3.95		11.0
649	2.43		6.2	2.25		10.3	2.80		10.4	3.61		11.1	3.80		13.4	3.24		10.2	2.01		6.4	1.53		5.8	1.44		7.3	1.44		8.5	1.44		7.3	1.44		7.1
650	2.43		6.2	2.25		10.3	2.80		10.4	3.61		11.1	3.80		13.4	3.24		10.2	2.01		6.4	1.53		5.8	1.44		7.3	1.44		8.5	1.44		7.3	1.44		7.1
651	1.44		4.6	1.60		8.8	2.15		5.2	2.23		4.3	2.90		8.4	2.90		5.8	1.92		4.6	2.04		5.4	2.55		7.1	2.75		8.2	2.18		7.4	1.60		3.8
651-A	2.12		6.1	1.98		7.3	2.64		7.7	2.97		2.9	3.32		7.2	3.42		9.2	2.56		6.3	2.05		10.7	2.90		7.4	2.72		9.2	2.11		6.4	2.03		5.4
652	2.05		4.6	1.76		6.5	2.43		7.7	2.66		4.5	2.91		6.4	2.14		7.5	2.66		6.1	1.48		4.0	1.98		6.4	2.27		10.1	4.06		9.1	3.46		10.3	2.37		6.8
653	2.05		4.6	2.01		5.9	2.83		6.6	2.91		6.4	2.14		7.5	2.66		6.1	1.48		4.0	1.98		6.4	2.27		10.1	4.06		9.1	3.46		10.3	2.37		6.8
654	3.25		8.9	2.67		8.6	2.83		6.6	3.04		6.8	3.32		8.2	3.42		9.2	2.56		6.3	2.05		10.7	2.90		7.4	2.72		9.2	2.11		6.4	2.03		5.4
655	3.59		7.6	2.69		6.5	2.65		7.4	3.06		6.5	2.38		4.3	1.38		5.7	0.50		2.2	0.87		2.9	2.90		6.8	4.98		8.2	4.06		10.3	4.50		9.6
656	2.98		22.5	2.51		12.9	3.18		14.7	4.72		10.5	2.38		5.3	1.65		5.2	0.78		4.8	1.28		3.8	2.80		6.2	6.72		20.6	6.18		13.8	7.68		17.0
657	2.62		12.5	2.38		7.9	1.95		7.4	1.54		5.3	1.11		3.1	0.49		3.1	0.27		1.7	0.49		2.1	1.51		4.5	3.29		8.5	3.75		10.6	2.91		7.3			
658	2.62		12.5	2.38		7.9	1.95		7.4	1.54		5.3	1.11		3.1	0.49		3.1	0.35		1.6	0.49		2.0	1.51		4.5	3.29		8.5	3.75		10.6	2.91		7.3			
659	4.24		10.9	4.14		12.8	4.65		15.5	4.35		10.0	3.64		7.6	3.52		9.8	2.06		8.3	1.06		7.1	1.06		4.1	2.24		6.9	2.24		6.9	2.24		6.9			
660	2.06	2.8		2.14	2.9		2.94	4.3		2.61	2.5		2.40	2.8		2.01	2.9		1.01	2.6		1.01	2.6		2.22	2.6		5.96	6.5	12.4	4.53	3.7		3.34	3.3		
661	2.21	1.8		2.14	2.1		1.85	2.4		2.10	1.6		2.10	1.6		2.10	2.1		2.10	2.1		2.10	2.1		2.41	4.0	11.3	3.85	6.7	12.4	2.83	5.0	10.5	2.13	2.6	9.4
662	1.30	0.9	4.9	1.36	2.4	4.9	1.88	2.2	4.9	2.40	3.2	6.5	2.82	3.1	10.0	3.01	2.8	8.1	2.68	2.6	6.1	2.85	4.1	9.6	2.02	3.4	8.3	3.70	4.3	11.2	2.43	3.4	7.3	1.86	1.9	
663	2.71	1.8		2.76	2.6		3.00	3.2		3.00	3.0		3.00	3.0		3.00	3.0		3.00	3.0		3.00	3.0		3.00	3.0		3.00	3.0		4.62	4.6		2.64	1.6		2.67	1.2	
664	2.71	1.8		1.97	1.0		2.27	1.3		1.84	1.1		2.27	1.3		2.80	2.4		2.78	1.6		2.38	1.9		2.53	4.4		2.96	2.0		2.64	1.4		2.67	1.2		
665	1.52	0.9	3.9	1.35	1.0	4.5	1.31	1.1	3.6	1.22	1.1	4.5	1.83	1.9	4.4	1.88	1.5	6.7	2.10	1.3	5.4	1.90	1.6	6.9	1.74	2.2	4.7	1.84	2.0	6.2	1.63	1.1	4.5	1.56	0.9	5.4
666	1.25	1.4		1.51	1.3		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5		1.50	1.5	
667	1.25	1.4		1.51	1.3		1.50	1.5		5.98	3.2		5.98	3.2		5.98	3.2		5.98	3.2		5.98	3.2		5.98	3.2		5.98	3.2		5.98	3.2		6.43	3.0	
668	2.36	4.9	10.0	1.74	2.2	7.2	1.97	3.5	7.0	1.85	2.1	6.4	2.26	1.8	8.4	1.55	3.5	7.8	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0
669	2.36	4.9	10.0	1.74	2.2	7.2	1.97	3.5	7.0	1.85	2.1	6.4	2.26	1.8	8.4	1.55	3.5	7.8	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0	2.10	4.0	8.0
670	3.31		14.0	2.18		4.9	2.46		5.2	2.72		5.9	2.62		5.1	3.06		7.0	2.50		6.7	2.24		6.2	2.24		6.2	2.24		6.2	2.24		6.2	2.24		6.2			
671	1.26	1.3	4.8	1.46	2.3	4.6	2.01	3.3	5.5	2.10	2.6	7.3	2.60	2.3	6.6	1.70	2.2	8.5	1.26	1.7	6.4	1.28	2.4	7.1	2.56	2.4	11.4	2.32	2.6	6.5	1.22	1.0	4.5	0.71	1.0	2.2
672	0.79	0.6	3.7	1.10	1.4	4.7	1.62	3.3	5.5	1.18	1.2	6.0	1.66	2.3	4.6	1.31	1.6	6.3	1.00	1.2	4.6	1.00	1.2	5.4	1.00	1.2	5.4	1.00	1.2	5.4	1.00	1.2	5.4	1.00	1.2	5.4
673	1.51		3.7	1.35		4.7	1.74		4.7	2.05		7.0	1.30		4.2	1.54		5.1	0.87		3.0	1.38		6.6	1.38		13.1	1.30		13.1	1.34		4.7	1.29		13.0			
674	1.51		3.7	1.35		4.7	1.74		4.7	2.05		7.0	1.30		4.2	1.54		5.1	0.87		3.0	1.38		6.6	1.38		13.1	1.30		13.1	1.34		4.7	1.29		13.0			
675	1.51		3.7	1.35		4.7	1.74	

TABLE 1.—

Station No.	Station.	Elevation above sea, in feet.	Years of Record.		Maximum Observed Precipitation.					Annual Precipitation.			Observed Precipitation, in Percentages of Mean Annual Rainfall.									Main references.
			Qur. Jury or less.	For month and year.	Last year tabulated.	Number of Minutes.			Day of 24 hours.	Month.	Minimum.	Mean.	Maximum.	Mean daily interval of rainfall.	Maximum During Short Periods.			Mean for month of maximum record.	Maximum.		Annual.		
						5.	15.	60.							Number of Minutes.				In 24 hours.	In one calendar month.	Minimum.	Mean.	Maximum.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
703	Dublin, Ireland	47	51	51	1921				3.4	7.6	16.6	27.4	38.5								100	140	25
704	Clyde Harbor, Ireland	30		35	1915							41.2									100		
705	Valencia, S. W. Ireland	30	52	52	1921				2.8	11.4	42.5	59.8	69.1					11.5	4.9	15.4	75	100	122	25; 4
706	Killarney, Ireland	174		35	1915							54.1									100		25
707	Kilkenny, Ireland	212		35	1915							33.1									100		25
708	Mullingar, Ireland	300		35	1915							36.7									100		25
709	Crossmolina, N. W. Ireland	67		35	1915							53.3									100		25
710	Armagh, N. E. Ireland	204	51	51	1921				2.6	7.2	23.6	31.7	39.6					9.9	8.2	22.7	74	100	125	
711	Belfast, N. E. Ireland	61		35	1915							34.5									100		25
712	Douglas, Isle of Man	277		46	1921				4.7	10.4	27.7	41.8	54.5					11.4	11.2	25.0	66	100	130	25
713	Stonyhurst, N. W. England	375	52	52	1921	1.25			2.5	12.7	30.6	46.5	60.5					8.2	5.4	27.3	66	100	130	25; 5
714	Scathwaite, N. W. England	422		35	1915							129.5									100		25
715	The Skye, N. W. England	1 070		35	1915							170.9									100		13
716	Glasgow, Scotland	180	51	51	1920				2.7	9.8	29.0	37.2	61.6					8.0	7.3	26.4	78	100	166	25
717	Ben Nevis, Scotland	4 406	20	20	1904				7.3	48.3	107.8	160.8	240.1					11.9	4.5	30.9	67	100	150	25
718	Fort William, W. Scotland	171	46	46	1921				4.5	23.1	61.2	91.1	108.3					13.0	5.8	29.7	25	100	140	25
719	Stornoway, Hebrides Islands	51	49	49	1921				2.8	14.9	32.2	49.8	71.6					12.6	5.6	30.0	65	100	144	25
720	Deerness, Orkney Islands	160	49	49	1921				3.6	8.2	29.1	35.4	44.6					11.1	10.2	23.2	82	100	128	25
721	Inverness, N. Scotland	98		35	1915							26.9									100		25
722	Aberdeen, Scotland	46	36	36	1915				3.8	9.4	23.9	30.4	43.8					10.7	9.2	31.0	77	100	144	25
723	Dundee, Scotland	160		35	1915							27.4									100		25
724	Crieff, Scotland	432		35	1915							40.9									100		25
725	Edinburgh (Leith), Scotland	30	50	50	1920				3.5	7.2	15.9	23.3	37.5					10.3	12.0	30.9	68	100	161	25
726	Wolfelee, Cheviot Hills, Scotland	537		35	1915							37.8									100		25
727	Tynemouth, N. E. England	66	51	51	1921				3.2	7.8	18.6	24.5	40.9					12.3	13.0	31.8	76	100	167	25
728	Liverpool, W. England	138		35	1915							27.9									100		25
729	Derby, Central England	992	47	47	1921				2.3	10.3	32.4	48.4	65.5					10.2	5.8	21.3	67	100	135	25
730	Yarmouth, E. England	17	34	34	1921				2.9	8.6	19.4	24.5	44.2					12.0	19.6	35.0	79	100	160	25
731	Vlieland, N. Holland	26		26	1910				2.1	7.5	10.4	22.8	33.4					14.4	9.5	33.0	46	100	147	27
732	Leeghwater, Holland	45		45	1911				2.3	10.3	21.6	32.1	46.6					13.0	7.2	31.0	66	100	145	27
733	Utrecht-De Bilt, Holland	19	63	63	1911				2.9	9.5	17.7	28.3	41.4					10.1	10.2	29.7	63	100	144	4; 27
734	Ostend, Belgium	12		50	1900							25.7									100		9
735	Brussels, Belgium	187		50	1900							28.8									100		9
736	Bouillon, Belgium	774		50	1900							47.4									100		9
737	Trevez, W. Germany	460		101	1924					9.4	15.1	26.9	39.2					8.3		34.9	56	100	146	4; 9
738	Freiburg, S. W. Germany	925		50	1900							33.4									100		9
739	Frankfort-am-Main, Germany	335		86	1924					5.4	14.1	20.5	36.8					10.9		34.5	56	100	151	4
740	Kiev, Russia	249		87	1924					6.2	19.1	30.0	38.4					9.3		27.3	64	100	126	4
741	Bremen, Germany			77								23.4									100		9
742	Berlin, Germany	115		74	1924					9.0	14.2	22.7	30.0					13.1		39.6	63	100	132	4
743	Dresden, Germany		27	58	1885				4.0	9.0	15.9	23.6	32.6					13.6	17.5	35.0	64	100	140	15
744	Breslau, Germany	462		50	1924					9.0	15.9	22.6	32.6					12.6		39.8	70	100	145	4
745	Prague, Czecho-Slovakia			50								18.5										100		20
746	Krzsica, Poland	1 923		44	1920						13.0	21.2	33.4	48.4				11.1		35.9	52	100	145	4
747	Lemberg, Poland	978		45	1920						16.6	20.3	27.9	37.7				15.0		45.0	74	100	136	4
748	Warsaw, Poland	426	40	100	1920				3.4	5.8	14.6	21.7	46.5	0.8				14.9	16.2	32.3	69	100	220	20; 4
749	Konigsberg, B. Prussia	10		84	1924					9.8	12.9	20.5	33.3					13.3		48.0	63	100	163	4
750	Vilna, E. Poland	486		43	1915						8.1	10.6	20.4	28.7				15.7		39.6	54	100	141	4; 20
751	Riga, Latvia	66	30	31	1882				1.7	7.4	12.5	20.9	30.4	0.7				10.5	8.5	37.0	62	100	152	20
752	Tallinn, Esthonia	46	37	40	1891				3.0	7.9	7.1	19.2	27.7					14.3	10.4	39.1	37	100	144	20; 21
753	Leningrad, Russia	16	44	95	1920				2.9	7.3	12.3	19.2	29.3	0.6				14.7	12.0	38.0	63	100	153	20; 4
754	Moscow, Russia	526	27	64	1916				2.1	5.7	9.0	13.7	22.7	32.8	0.8			9.6	7.3	35.2	61	100	144	20; 4
755	Gorki, Russia	679	32	42	1891				3.1	8.1	12.9	19.9	28.5	0.6				13.7	15.6	40.7	64	100	151	20; 21
756	Kiev, Russia	600	23	63	1915				4.1	8.3	15.0	22.5	32.5	0.8				14.1	18.5	39.7	59	100	151	29; 4
757	Odessa, Russia	213	20	69	1915				3.1	6.6	9.0	15.5	24.9	1.3				12.8	18.9	42.4	56	100	164	22; 23; 22
758	Tagarog, Russia	115		30	1915				5.5	6.6	9.0	15.5	26.4					12.8	34.1	42.2	58	100	164	22; 20
759	Sevastopol, Crimea	1132	16	36	1882				2.1	4.7	8.6	15.2	25.3	1.0				9.2	32.5		55	100		22
		1 76	30	30	1908				4.6			14.4						7.8	32.5			100		22
760	Novorossisk, Russia	122	34	40	1914				4.0	10.3	11.7	27.1	45.5					12.3	14.6	37.6	43	100	168	4; 22; 20
761	Batum, Trans-Caucasia	10		26	1906				10.3	20.5	66.3	98.3	111.0					9.5	10.5	20.9	68	100	113	22; 26
762	Tiflis, Trans-Caucasia	1 523		70	1914					5.0	11.1	19.6	30.4					15.0		45.0	57	100	156	4
763	Lenkoran, Trans-Caucasia	66		27	1891					19.6	29.3	46.5	66.5	1.0				17.7		42.2	63	100	143	20; 21
764	Baku, Trans-Caucasia	1	20	39	1891				4.1	7.7	9.0	13.7	41.3	0.5				13.7	41.3	68.4	51	100	146	20; 21
765	Astrakhan, Russia	49	38	68	1915				2.2	6.0	1.0	6.3	12.4	2.2				8.3	35.9	95.5	16	100	196	20; 4
766	Logan, S. Russia	164	43	43	1893				2.5	5.1	7.1	12.5	20.9	0.9				9.9	16.8	48.2	59	100	144	20; 21
767	Nikolaevsk, Russia	633		36	1915					6.0	10.2	14.5	22.6	1.2				14.3		41.5	70	100	155	4; 20
768	Kasan, E. Russia	266		54	1916					6.0	9.2	14.8	22.1					14.5		41.1	65	100	151	4; 20
769	Blatoust, E. Russia	1 476	44	44	1891				3.5	10.2	8.8	18.6	31.4	0.6				15.8	18.8	54.8	46	100	168	20; 21
770	Orenburg, E. Russia	276	32	64	1916				1.7	6.5	9.2	15.0	24.0	0.9				12.5	11.6	43.0	61	100	160	4
771	Irgiz, W. Siberia	367	18	26	1891				1.2	3.5	2.8	6.9	9.6	2.0				9.9	17.4	50.8	41	100	139	20; 21
772	Khiva, Turkestan	328		17	1891					2.3	0.8	2.1	3.4					26.6		109.4	28	100	162	20; 21
773	Tashkend, Turkestan	2 376		20	1891					2.3	0.8	2.1	3.4	2.8				19.0		109.4	28	100	137	20; 21
774	Barnaul, S. Siberia	479	43	54	1891				2.2	5.8	4.2	11.5	17.7	1.2				14.3	19.1	50.4	37	100	154	20; 21
775	Irkutsk, S. Siberia	1 509		17	1891					7.1	9.8	14.5	18.8	1.6				15.8		49.0	66	100	130	20; 21
776	Nertschinsk, S. Siberia	2 153	42	51	1891				6.2	9.1	9.6	15.8	24.1	1.6				26.6	38.7	49.0	66	100	162	20; 21
777	Nikolaevsk-on-Amur, S. Siberia	108		60	1919				2.5	9.0	7.8	17.6	30.3	1.2				15.2	14.2	51.0	44	100	172	17
778	Ajan, E. Siberia	21	13	13	1919				3.5	9.3	10.0	18.2	35.0	1.3				22.3	10.3	54.0	60	100	231	17
779	Okhotsk, E. Siberia	20	23	23	1919				1.9	11.2	6.4	11.2	16.5					20.6	16.9	100.9	67	100	147	17
780	Gishiga, E. Siberia	40		24	1919					4.2	3.2	12.3	42.5					20.3		133.0	33	100	133	17
781	Petropaulovsk, E. Siberia (S. Kamchatka)	40		8	1889					16.1	36.4	47.0	76.0					12.4		34.5	78	100	162	20; 21
782	Navo-Martinski, N. E. Siberia	16		21	1919					4.3	4.7	7.4	10.3					20.6		88.1	64	100	139	17
783	Markoyo, N. E. Siberia	66	25	26	1919				1.8	9.1	5.6	7.9	10.9					24.1	30.8	77.2	64	100	127	17
784	Verkhoyansk, N. Siberia	326	34	34	1919				0.8	2.4	2.0	5.0	7.5					17.6	16.0	68.0	40	100	182	17
785	Yakutsk, Siberia	334	26	26	1921				1.4	4.1	3.6	6.2	12.3	0.7				14.5	18.9	62.3	54	100	173	17
786	Vilujsk, N. Siberia	394	26	26	1923				1.8	4.9	5.7	8.9	11.5					18.0	20.6	61.0	54	100	158	17
787	Turuchansk, N. W. Siberia	131	29	29	1921				1.4	5.1	4.0	8.0	12.5	0.7				19.9	10.8	43.2	38	100	142	17
788	Obdorsk, N. W. Siberia	59		27	1909					5.8	4.0	10.1	14.4	1.1				20.9		67.3	44	100	142	17
789	Berezov, N. W. Siberia	134	28	28	1909				2.6	6.7	7.1	12.5	18.5					16.5	18.5	61.7	41	100	147	17
790	Surgut, N. W. Siberia	138	24	24	1909				2.7	6.0	11.8	15.8	23.0					16.4	17.0	37.8	74	100	145	17
791	Bogosirsk, W. Siberia	623	42	42	1891				2.6	7.7	7.1	16.0	27.4	0.8				14.4	16.8	47.0	33	100	161	20; 21
792	Ekaterinburg, W. Siberia	921	45	80	1915				2.7	8.4	6.9	15.2	24.4	1.0				19.0	24.3	55.4	45	100	160	20; 4
793	Perm, E. Russia	534		33	1915					7.3	17.6	23.2	31.4	0.7				12.3		31.9	76	100	135	4
794	Ust-Kylma, N. E. Russia	203		51	1916					5.1	9.6	16.6	29.3					14.3		30.5	66	100	177	4; 20
795	Archangel, N. Russia	22		51	1916					5.1	9.6	16.6	29.3	1.0				14.3		30.5	66	100	177	4; 20
796	Kem, N. W. Russia	20	18	28	1891				1.8	4.1	3.6	7.9	12.4					12.5	12.4	33.0	70	100	150	20; 21
797	Helsingfors, Finland	58		50	1924					7.6	14.3	24.4	33.9					9.2		31.1	59	100	139	4
798	Abo, S. W. Finland	66		49	1922					7.3	15.7	23.8	33.0					12.5		30.6	66	100	136	20
799	Upsala, Sweden	79		74	1924					7.9	12.3	21.3	32.0					13.6		37.4	58	100	151	4
800	Ostersund, Sweden	1 017		51	1924					6.3	11.2	19.6	26.4					11.6		33.9	48	100	157	4
801	Haparanda, Sweden	20		46	1924					7.7	9.6	18.3	30.1					11.6		33.9	48	100	157	4
802	Karesuanda, Sweden	1 092		46	1924					6.1	6.8	12.4	20.2					18.9		49.1	55	100	163	4
803	Kols, Lapland	33		14	1891					2.4	4.9	6.9	10.6					15.6		34.8	71	100	154	21; 20
804	Gjesvar, N. Norway	26		37	1920					10.4	17.3	29.1	46.1					11.4		35.6	59	100	158	4
805	Bodo, N. Norway	76		93	1920					12.4	30.7	45.3	68.2					11.4		36.0	65	100	143	4
806	Namsian, S. Norway	85		55	1920					8.1	14.5	22.1	31.7					10.6		30.6	65	100	143	4
807	Copenhagen, Denmark	16		100	1920					8.0	10.4	22.5	30.3					7.9		35.5	46	100	136	4
808	Lerwick, Shetland Islands	113	50	50	1921				2.8	10.8	27.6	38.0	57.3					11.2	7.3	28.4	72	100	150	25
809	Thorshavn, Faroes Islands	89		48	1920					13.7	37.6	57.2	77.1					11.5		33.9	66	100	134	4
810	Berufjord, E. Iceland	64		51	1923					15.1	32.5	45.9	68.4					10.8		32.9	49	100	149	4
811	Vestmanna, S. W. Iceland	37		43	1922					12.9	27.2	38.7	70.7					10.8		25.8	69	100	131	4
812	Stykkisholm, N. W. Iceland	98		67	1923					9.6	15.8	25.7	40.5					10.0		33.4	54	100	157	4
813	Grimsey, N. Iceland	76		51	1923					7.3	12.1	19.1	27.1					11.6		42.0	49	100	163	4
814	Angmagssalik, E. Greenland	109		37	1920					18.6	21.8	35.8	58.4					17.6		52.2	61	100	164	4
815	Upernivik, N. W. Greenland	68		47	1920					6.8	8.4	9.0	17.8					10.1		69.0	38	100	198	4
816	Jacobshavn, W. Greenland	46		47	1920					7.6	8.3	9.1	17.6					15.2		65.0	37	100	196	4
817	Godthab, S. W. Greenland	35		37	1920					11.0	13.2	23.7	36.0					13.1		46.5	51	100	150	4
818	Ivigtut, S. Greenland	31		45	1920					20.6	20.9	44.8	72.8					9.6		46.0	47	100	164	4
819	Horta, Azores	210		22	1923					9.9	20.5	42.0	64.7					11.2		33.0	71	100	149	4
820	Ponta-Delgada, Azores	133		50	1923					9.5	19.6	28.3	44.2					8.2		33.5	69	100	149	4

(Continued.)

Station No.	January.			February.			March.			April.			May.			June.			July.			August.			September.			October.			November.			December.		
	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.	Mean month.	Maximum. 24 hours.	Maximum. Month.
	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)	(56)	(57)	(58)	(59)	(60)	(61)
103	2.29	1.8	5.7	1.89	1.6	3.8	1.84	1.3	3.7	1.90	1.6	4.7	2.05	2.0	5.5	1.95	1.8	6.2	2.50	2.0	6.3	3.04	3.4	7.6	1.92	2.1	5.1	2.68	2.8	7.4	2.67	2.0	6.2	2.48	1.8	7.6
104	4.15			3.70			3.01			2.58			2.42			2.67			2.85			3.76			2.97			4.09			4.20			6.32		
105	6.87	2.8	10.6	5.07	1.9	11.0	4.38	1.7	10.8	3.31			3.09			3.71	1.7	7.8	3.84	1.9	9.7	4.83	2.3	8.5	4.49	2.0	10.0	5.37	2.5	10.4	5.64	2.7	9.7	6.50	1.9	11.4
106	6.87			5.07			4.38			3.31			3.09			2.91			3.32			4.42			2.58			5.37			5.63			7.28		
107	3.20			2.54			2.29			2.18			2.42			2.45			2.60			3.41			2.31			3.16			3.46			4.05		
108	3.20			2.54			2.29			2.18			2.42			2.45			2.60			4.17			2.31			3.16			3.46			4.05		
109	3.24			4.46			4.52			3.04			3.04			3.04			3.04			3.62			3.62			3.62			3.62			3.62		
110	2.89			2.22	1.2	4.5	2.55	1.1	4.4	2.23			2.01	1.3	4.0	2.23			2.23			2.66			2.66			2.66			2.66			2.66		
111	2.89			2.22			2.55			2.23			2.01			2.23			2.23			2.66			2.66			2.66			2.66			2.66		
112	3.39	2.0	7.0	3.27	1.3	7.2	2.97	1.7	5.5	2.47	1.1	5.0	2.53	2.0	6.0	2.47	1.6	4.9	3.06	1.8	6.6	3.59	2.2	7.7	3.30	4.7	9.7	4.62	2.2	9.2	4.78	2.6	10.4	6.01	2.0	6.3
113	4.28	2.1	8.6	3.35	2.0	6.8	3.69	1.5	7.1	2.71	1.2	5.7	2.89	1.9	6.5	2.97	1.7	8.7	3.85	2.2	8.5	5.00	1.9	9.8	3.82	2.0	12.7	4.49	2.5	10.8	4.81	2.2	8.2	4.85	1.5	10.6
114	13.28			11.87			11.15			7.42			7.37			6.52			8.40			11.57			9.92			12.00			13.56			16.24		
115																																				
116	3.27	1.3	7.4	2.89	2.7	9.2	2.67	1.1	6.8	2.08	1.2	4.4	2.59	1.1	4.7	2.52	2.4	9.1	3.03	2.1	6.5	3.91	2.6	6.9	3.86	1.6	9.8	3.40	2.6	8.2	3.64	1.6	6.5	4.15	1.3	7.4
117	18.40	8.3	35.2	13.55	6.7	36.2	15.24	5.5	37.8	8.40	4.4	20.2	7.81	3.8	15.7	6.67	3.1	13.3	7.22	3.4	14.0	10.15	4.2	12.0	6.24	2.2	14.0	7.09	4.5	14.1	6.07	2.3	10.2	6.76	2.3	13.1
118	9.58	3.7	20.7	7.46	3.2	19.1	6.96	2.2	15.9	4.43	2.6	9.7	3.89	1.7	6.7	3.56	1.6	11.1	4.85	3.3	11.0	6.14	2.4	12.0	5.26	2.2	8.5	5.15	2.1	9.6	5.83	1.9	9.6	6.25	1.6	16.9
119	5.17	2.6	9.0	4.46	1.6	8.0	4.10	1.5	8.1	3.03	1.2	6.8	2.56	1.9	5.2	2.32	1.3	5.4	3.03	1.7	6.8	3.97	2.2	8.5	2.95	1.9	8.5	3.15	2.1	9.6	3.93	1.6	8.2	4.39	1.7	7.6
120	3.45	1.3	5.9	3.01	1.7	5.9	2.81	1.3	5.8	2.07	1.1	4.2	1.99	1.8	3.8	1.94	1.0	3.7	2.57	1.6	5.2	2.87	3.6	6.9	2.90	1.6	6.0	3.79	1.7	7.1	3.93	1.6	8.2	4.39	1.7	7.6
121	2.48			2.16			3.13			1.59			1.81			1.87			2.49			2.26			2.50			2.60			2.50			2.60		
122	2.48	4.1		2.10	1.5	4.7	2.52	1.4	5.2	1.99	1.5	4.8	2.25	1.1	4.2	1.89	1.0	4.0	2.91	1.5	5.9	2.94	1.9	7.0	2.42	2.5	6.2	3.01	1.9	6.2	3.17	2.8	6.2	3.24	3.5	9.4
123	1.95			1.86			2.09			1.70			2.08			1.80			2.74			3.38			2.08			2.44			2.66			2.66		
124	4.03			3.52			3.29			2.19			2.49			2.64			2.97			4.21			2.86			3.93			4.34			4.65		
125	1.66	1.4	4.5	1.42	1.0	5.3	1.75	1.0	4.4	1.28	1.2	4.3	1.83	1.4	5.3	1.73	1.4	4.7	2.55	2.7	5.6	2.94	2.8	6.0	1.76	1.4	5.0	2.41	2.0	7.2	1.96	1.3	5.2	2.00	1.0	5.7
126	3.24			2.77			3.52			2.26			2.24			2.34			3.08			3.34			2.67			3.37			4.30			4.65		
127	1.63	0.9	4.2	1.57	2.0	4.3	1.42	1.3	3.9	1.99	2.5	5.9	1.98	2.6	5.4	2.40	1.7	5.4	2.26	2.4	6.1	1.80	1.6	5.0	2.01	3.2	7.8	2.13	2.4	5.8	2.18	1.3	5.6
128	2.15			1.98			1.99			1.94			2.01			2.02			2.41			2.37			2.50			2.60			2.65			2.65		
129	4.47	1.9	9.7	3.91	2.7	9.7	3.91	2.7	9.7	3.91	2.7	9.7	3.91	2.7	9.7	3.93	2.0	7.9	4.56	1.8	8.2	3.93	2.6	5.9	3.93	2.2	8.6	7.41	3.0	8.2	2.44	2.5	5.9
130	1.74	1.1	3.7	1.49	0.8	3.4	1.70	0.9	3.7	1.48	1.0	3.9	1.73	1.1	3.7	1.73	1.2	3.7	1.98	1.6	5.4	2.76	3.1	6.3	2.29	1.2	7.5	2.29	1.2	9.6	2.01	0.9	4.9
131	1.58	0.6	4.0	1.34	0.7	3.3	1.59	0.7	3.1	1.24	0.9	3.2	1.47	1.1	3.4	1.40	1.6	3.4	1.72	1.4	5.6	2.76	3.1	6.3	2.29	1.2	7.5	2.29	1.2	9.6	2.01	0.9	4.9
132	2.13	1.2	4.0	1.72	1.4	4.8	1.97	1.0	4.1	1.71	1.2	4.1	1.96	1.4	5.2	2.02	1.7	5.1	2.33	1.7	6.2	3.11	2.7	6.9	2.56	1.4	8.0	2.89	1.5	8.6	2.38	1.5	4.7	2.71	1.7	6.0
133	1.90			1.78			1.89			1.82			2.33			2.56			4.03			3.77			3.69			4.96			4.92			3.48		
134	4.35			2.94			3.51			2.77			3.56			3.56			4.03			3.77			3.69			4.96			4.92			3.48		
135	1.94		5.0	1.67			1.89	4.9	5.4	1.74	5.1	2.27	2.56	6.3	6.1	2.79	6.5	6.5	2.64	6.5	9.4	2.44	7.1	8.33	2.44	7.1	2.65	6.1	6.1	2.65	6.1	6.1
136	1.65			1.67			2.05			2.72			3.40			4.13			3.05			3.34			2.65			3.37			4.30			2.65		
137	1.76	4.8		1.67	4.5		2.05	4.2	5.7	2.72	5.7	3.40	4.13	7.1	7.9	3.05	8.2	8.2	2.64	8.4	8.4	1.86	4.3	6.2	2.15	6.2	6.2	1.89	6.0	6.0	2.05	4.2	4.4
138	1.96	3.4		1.94	3.4		2.05	3.4	5.7	2.72	5.7	3.40	4.13	7.1	7.9	3.05	8.2	8.2	2.64	8.4	8.4	1.86	4.3	6.2	2.15	6.2	6.2	1.89	6.0	6.0	2.05	4.2	4.4
139	1.74	1.1	3.7	1.49	0.8	3.4	1.70	0.9	3.7	1.48	1.0	3.9	1.73	1.1	3.7	1.73	1.2	3.7	1.98	1.6	5.4	2.76	3.1	6.3	2.29	1.2	7.5	2.29	1.2	9.6	2.01	0.9	4.9
140																																				
141	1.07	3.9	1.41	4.6	1.68	4.3	5.1	1.52	4.3	1.91	4.3	2.35	2.99	2.97	6.4	3.21	7.2	7.2	2.65	7.7	9.0	2.44	1.6	7.7	1.87	3.5	5.0	1.49	2.2	4.6	1.62	1.8	3.6	1.54	1.2	4.3
142	1.21	0.7	3.9	1.23	0.7	3.7	1.44	0.8	4.0	1.25	0.9	3.2	1.55	1.1	3.9	1.57	1.2	3.9	2.03	1.2	5.2	2.65	2.0	6.0	1.53	3.7	6.0	1.53	3.7	6.0	1.46	1.4	4.0	1.49	1.2	4.3
143	1.27		2.9	1.25		3.5	1.80		3.9	1.40		3.6	1.77		4.2	2.28		6.0	2.46		6.6	2.42		6.9	1.69		4.2	1.42		4.0	1.42		4.0	1.42		4.0
144																2.63																				
145	0.63																																			
146	2.36			1.67		4.1	2.05		4.2	2.28		6.3	3.23		6.1	4.00		10.9	4.69		10.8	3.70		12.0	2.99		7.6	2.44		6.9	1.73		3.9	2.13		4.4
147	1.26		3.1	1.20		3.0	1.62		4.3	1.81		5.0	2.68		6.6	3.32		10.0	3.23		8.2	2.17		7.5	1.66		3.5	1.67		5.4	1.47		3.4	1.46		2.9
148	1.21	0.6	3.0	1.08	0.6	2.8	1.35	0.9	3.2	1.49	1.2	3.9	1.89	1.5	4.6	2.57	2.0	4.3	2.12	3.4	6.8	2.90	2.4	5.5	1.90	1.9	3.5	1.67	1.4	5.4	1.47	1.1	4.2	1.46	0.8	2.9
149	1.27		4.3	1.05		3.4	1.04		3.3	1.07		3.6	1.46		5.0	1.85		5.6	2.26		7.9	2.72		9.8	2.46		7.1	1.91		7.0	1.80		6.3	1.59		5.9
150	1.02		3.0	0.88		2.6	0.99		2.2	1.15		2.8	1.93		6.4	2.54		5.6	2.66		5.9	3.20		8.1	1.89		2.9	1.53		3.5	1.31		3.1	1.16		3.6
151	1.20	0.6	2.8	0.81	0.7	2.0	1.00	0.7	2.3	1.09	0.6	2.5	1.64	1.5	3.0	1.60	1.3	4.0	2.00	1.9	4.5	2.75	2.0	7.5	2.18	2.0	6.6	2.14	1.4	6.5	1.84	1.5	4.1	1.39	0.7	5.2
152	1.01	0.7	2.1	0.83	0.6	1.9	0.95	0.7	3.5	0.95	0.6	2.5	1.64	1.5	3.0	1.60	1.3	4.0	2.00	1.9	4.5	2.75	2.0	7.5	2.18	2.0	6.6	2.14	1.4	6.5	1.84	1.5	4.1	1.39	0.7	5.2
153	0.94	0.6	2.6	0.88	0.8	2.3	0.89	1.1	2.2	1.03	0.8	2.3	1.64	1.2	4.4	1.99	2.0	5.2	2.82	2.3	7.3	2.14	3.8	7.0	1.76	1.0	4.1	1.60	0.7	3.5	1.23	1.3	3.7
154	1.30	0.7	3.4	1.12	0.7	2.6	1.20	0.6	2.6	1.30	0.7	2.9	1.90	1.4	4.0	2.34	1.5	7.0	3.05	1.4	6.5	2.90	1.7	6.4	2.17	1.8	5.6	1.66	1.6	6.8	1.38	1.0	3.3	1.07	0.7	4.3
155	1.30	0.7	3.4	1.12	0.7	2.6	1.20	0.6	2.6	1.30	0.7	2.9	1.90	1.4	4.0	2.34	1.5	7.0	3.05	1.4	6.5	2.90	1.7	6.4	2.17	1.8	5.6	1.66	1.6	6.8	1.38	1.0	3.3	1.07	0.7	4.3
156	1.30	0.7	3.4	1.12	0.7	2.6	1.20	0.6	2.6	1.30	0.7	2.9	1.90	1.4	4.0	2.34	1.5	7.0	3.05	1.4	6.5	2.90	1.7	6.4	2.17	1.8	5.6	1.66	1.6	6.8	1.38	1.0	3.3	1.07	0.7	4.3
157	1.30	0.7	3.4	1.12	0.7	2.6	1.20	0.6	2.6	1.30	0.7	2.9	1.90	1.4	4.0	2.34	1.5	7.0	3.05	1.4	6.5	2.90	1.7	6.4	2.17	1.8	5.6	1.66	1.6	6.8	1.38	1.0	3.3	1.07	0.7	4.3
158	1.30	0.7	3.4	1.12	0.7	2.6	1.20	0.6	2.6	1.30	0.7	2.9	1.90	1.4	4.0	2.34	1.5	7.0	3.05	1.4	6.5	2.90	1.7	6.4	2.17	1.8	5.6	1.66	1.6	6.8	1.38	1.0	3.3	1.07	0.7	4.3
159	1.21	1.4	4.0	0.97	1.1	2.2	0.99	0.7	3.5	1.08	1.4	4.0	1.78	1.5	4.0	1.63	1.7	4.0	1.94	1.6	7.5	1.87	2.2	5.6	1.43	1.7	4.4	1.67	1.4	6.8	1.45	1.2	4.2	1.20	0.7	5.3
160	1.21	1.4	4.0	0.97	1.1	2.2	0.99	0.7	3.5	1.08	1.4	4.0	1.78	1.5	4.0	1.63	1.7	4.0	1.94	1.6	7.5	1.87	2.2	5.6	1.43	1.7	4.4	1.67	1.4	6.8	1.45	1.2	4.2	1.20	0.7	5.3
161	1.17	0.8	2.9	1.13	0.7	2.9	0.85	0.7	3.0	1.04	0.8	2.8	1.62	1.2	4.0	1.58	1.3	3.8	1.83	1.4	7.5	1.82	2.0	5.5	1.42	1.6	4.4	1.67	1.4	6.8	1.45	1.2	4.2	1.20	0.7	5.3
162	1.17	0.8	2.9	1.13	0.7	2.9	0.85	0.7	3.0	1.04	0.8	2.8	1.62	1.2	4.0	1.58	1.3	3.8	1.83	1.4	7.5	1.82	2.0	5.5	1.42	1.6	4.4	1.67	1.4	6.8	1.45	1.2	4.2	1.20	0.7	5.3
163	2.48	2.6	10.2	2.89	2.3	8.5	2.43	3.9	8.0	1.84	2.3	4.6	1.61	2.6	6.0	2.28	3.0	6.4	6.42	5.8	8.5	9.35	10.3	20.5	12.10	7.5	17.4	9.23	4.9	11.4	12.73	5.1	11.2	10.38	3.9	14.5
164																																				
165	10.11	3.6	12.4	7.14	2.9	11.6	5.89	2.9	13.8	4.93	2.9	6.7	3.20	3.9	6.7	3.26	3.9	6.3	6.73	6.3	7.2	1.96		5.3	1.51		7.8	2.00		7.8	1.44		6.6	1.21		4.2	0.89		5.3
166	0.58		2.8	0.78		2.1	3.46		7.4	2.48		5.5	1.47		4.0	1.01		3.0	1.46		4.0	1.15		3.4	1.78		4.8	1.19		6.4	0.86		4.2	0.64		3.2
167	4.56	1.7	9.4	3.41	1.2	10.0	2.67	1.1	6.8	2.08	1.2	4.4	2.59	1.1	4.7	2.52	2.4	9.1	3.03	2.1	6.5	3.91	2.6	6.9	3.86	1.6	9.8	3.40	2.6	8.2	3.64	1.6	6.5	4.15	1.3	7.4
168	1.33	0.6	3.6	0.87	0.2	2.5	0.85	0.4	3.1	0.51	0.6	2.2	0.70	0.2	2.7	0.73	0.1	6.2	0.90	0.2	5.1	0.77	0.1	4.2	0.77	0.1	4.2	0.77	0.1	4.2	0.77	0.1	4.2	0.77	0.1	4.2
169	0.58	0.4	2.8	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1
170	0.79	0.6	2.2	0.63	0.1	1.8	0.87	0.6	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
171	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.2	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
172	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.2	2.3	0.85	0.1	1.6	1.74	1.1	4.6	2.90	2.5	6.5	2.93	2.4	10.2	3.24	1.2	5.5	2.93	1.2	7.5	3.64	1.3	9.3	3.89	1.4	10.9	4.20	1.5	9.8
173	0.58	0.4	2.8	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1
174	0.79	0.6	2.2	0.63	0.1	1.8	0.87	0.6	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
175	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.2	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
176	0.58	0.4	2.8	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1
177	0.79	0.6	2.2	0.63	0.1	1.8	0.87	0.6	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
178	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.2	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
179	0.58	0.4	2.8	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1
180	0.79	0.6	2.2	0.63	0.1	1.8	0.87	0.6	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
181	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.2	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
182	0.58	0.4	2.8	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1
183	0.79	0.6	2.2	0.63	0.1	1.8	0.87	0.6	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
184	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.2	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
185	0.58	0.4	2.8	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1	0.46	0.2	2.1
186	0.79	0.6	2.2	0.63	0.1	1.8	0.87	0.6	2.3	0.85	0.1	1.6	1.58	0.4	4.6	2.11	0.6	6.0	1.13	0.3	3.8	1.63	0.4	4.5	1.32	0.4	4.1	1.18	0.4	4.3	0.86	0.3	3.2	0.64	0.2	3.2
187	0.72	0.6	1.6	0.58	0.1	1.7	0.72	0.

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PLASTIC FLOW IN CONCRETE ARCHES*

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Synopsis

Many materials of construction, especially concrete during the first few months after fabrication, do not follow the laws upon which the theory of elasticity in its many forms is based. Yet this theory has been persistently used and elaborated for indeterminate structures, regardless of the material. Such factors as the yield of a stressed body with time, often called "hysteresis", and the variation of the ratio of stress to elastic strain, when at all accounted for in a structural analysis, have generally been analyzed in some more or less unscientific method of guess. For concrete arches, especially in Europe, construction is frequently furthered, that is, by the introduction of temporary hinges, hydraulic jacks, etc., in such a manner that these indeterminate factors can be controlled during the construction and early life of the structure.

Obviously, the ordinary theory of elasticity is entirely inadequate for studying stresses in a statically indeterminate structure made of a material which, when under load, yields with time as well as stress and does not obey Hooke's law for elastic deformation. To fill this need, methods of analysis have been developed by the writer based on experiment. These methods provide a much more general statement of the displacements and deformations in structures than is given by the ordinary theory of elasticity because they fit a greater number of properties of materials. Further, they take the exact form of the usual theory of elasticity when plastic deformation is not considered and Hooke's law is assumed to apply. For convenience, the proposed methods of analysis have been termed the "theory of plasticity" and the "proposed theory of elasticity". Although these theories were developed in particular to investigate changes and variations of the stressed condition of concrete arches due to plastic flow, they are of general significance in structural design.

Note.—Written discussion on this paper will be closed in May, 1930, *Proceedings*.

* This paper was developed from a phase of an unpublished investigation made by the writer while a "Fellow" in Civil Engineering at the University of Illinois, 1925-27. The title of the manuscript is "Neglected Factors in the Analysis of Stresses in Concrete Arches".

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These investigations show definitely that one may be led astray in drawing conclusions regarding the stressed condition of an arch, even if based upon precise and accurate measurements of deformations. The studies indicate that it is even possible to find observed deformations to check the results of analysis by the ordinary elastic theory and yet the resulting conclusions might be wrong. Such misconceptions are readily verified by the procedures herein proposed.

Statements that a locally overstressed point in a structure relieves itself as a result of plastic flow, are shown to be unwarranted and misleading. A much clearer conception is given with regard to shrinkage stresses, temperature stresses, effects resulting from movement of arch abutments, etc. Valuable inferences may be drawn as to the proper interval before removing the arch centering.

Nomenclature

Ordinarily, the nomenclature for arches is quite involved. When, in addition, such factors as plastic flow and variable stress-strain relations are considered, necessarily it becomes very cumbersome. Hence, some duplication has been used, but it has allowed simplification, and should not lead to confusion. In most instances, H_0 has been used to designate the neutral point horizontal thrust due to all causes, that is, superimposed load, dead load, rib-shortening, etc.; in a few cases, it refers to the thrust resulting only from dead and live loads.

As much as practicable of the ordinary American nomenclature for arches has been used. Simplifications, or rather condensations, and additions have also been adopted to make cumbersome algebraic expressions easy to handle. As an example, the symbol, V, has been used to replace a frequently recurring algebraic expression, much the same as the ordinary substitution of I, for the moment of inertia of a section.

Subscripts denote reference points and causes; for example, $H_{0\text{-}RS}$ indicates the horizontal thrust considered to act through the neutral point, O, of an arch, due to rib-shortening under load. Prime marks (superscripts) over a letter denoting a variable distance indicate a definite distance to a particular point or line. Thus, while z indicates the distance from the centroidal axis of a section to any point in the plane of the section, z' refers to the distance from the centroidal axis of the section to the line of action of the thrust acting upon the section.

In general, distances are considered positive when measured upward or to the right. Similarly, external force components having their sense upward or to the right are positive.

Thrusts acting within a member are considered positive when they cause tension. Shear at a section in a horizontal or inclined member is positive when the section resists impending motion upward at the left portion of the structure. In general, moments in the rib of an arch are called positive when they increase the compression at the extrados and decrease it at the intrados.

Angles are measured positive in a counter-clockwise direction from the initial line.

Notation.—The following nomenclature is common to the analyses presented. The less common terms are not included, but will be defined as they are introduced into the discussion.

$d\xi$ = the shortening along the arch axis of a differential length of the rib.
$d\tau$ = the transverse or sliding movement within a differential length.
$d\phi$ = the angle change within a differential length.
ds = a differential length of a structure measured along the centroidal axis.
$\Delta t°$ = the total temperature change, in degrees Fahrenheit.
α = the angle between a tangent to the arch axis and the x-axis of co-ordinates.
β = the angle which the quasi-horizontal x-axis, for an unsymmetrical arch, makes with the horizontal.
θ = the change in slope of the tangent at a point on a structure.
γ = the total unit of shrinkage of a given concrete at a particular time.
$\delta_1, \delta_2, \delta_n$ = respectively, the displacement of points in a structure.
δ_{xR}, δ_{xL} = respectively, the permanent displacements of the right and left springings of an arch in the direction of the x-axis of co-ordinates.
δ_{yR}, δ_{yL} = respectively, the permanent displacements of the right and left springings of an arch in the direction of the y-axis of co-ordinates.
δ_{rR}, δ_{rL} = respectively, the permanent rotational displacements of the right and left springings of an arch.
ϵ = the unit elastic deformation.
ϵ_z = the unit elastic deformation at a distance, z, from the centroidal axis.
ϵ_u = the unit elastic deformation at a distance, u, from the center of elastic rotation.
ϵ_u' = the unit elastic deformation along the centroidal axis.
μ = the coefficient of expansion per degree Fahrenheit.
ρ = the unit of plastic deformation.
ρ_v = the unit of plastic deformation at a distance, v, from the center of plastic rotation.
σ = unit stress.
σ_z = unit stress at a distance, z, from the centroidal axis.
σ_u = unit stress at a distance, u, from the center of elastic rotation.
b = the width of a section.
c = the distance from the centroidal axis of a section to the outer fiber.
c_0 = the distance from the centroidal axis to the outer fiber for a normal section at the crown of an arch.
d = the thickness of a normal section.
d_0 = the thickness of an arch at the crown.
E = Young's modulus of elasticity.
f = rise of arch from springing to crown.
f', f'', f''', these terms have been used freely to designate functions and constant factors in terms, especially where only relative values are of importance.

H_0 = the reaction at the center of co-ordinates considered to act in line with the x-axis at the end of a rigid arm fixed to the arch at the springing.

$H_{0\text{-}DL}$ = the horizontal thrust considered to act through the neutral point due to the dead load, that is, neglecting all effects of rib-shortening, temperature change, or shrinkage.

$H_{0\text{-}RS}$ = the horizontal thrust considered to act through the neutral point as a result of rib-shortening due to the compression of the arch.

H_R = the horizontal reaction in a symmetrical arch, or quasi-horizontal reaction in an unsymmetrical arch, at the right support when the arch is considered as a simple beam.

$I = \frac{2}{3} bc^3$ = the moment of inertia of a normal section according to the ordinary theory of elasticity.

K = a parameter in the exponential equation defining plastic deformation.

k = a parameter in the exponential equation defining elastic deformation.

l = the span of the arch measured parallel to the x-axis.

l' = the vertical distance between springings of the arch measured parallel to the y-axis.

Δl = increment or change in l, due to deformation of the arch.

$\Delta l'$ = increment in l', due to deformation.

Δl_{RS}, $\Delta l'_{RS}$ = increments, respectively, in l and l' due to rib-shortening in the arch.

m = an exponent of stress determined from experiment and defining the rate of elastic deformation with change in stress. The number, m, may also appear in equations as a coefficient.

M = the bending moment.

M_0 = the bending moment on a lever arm extending from the springing to the center of co-ordinates.

M_x = the bending moment at a section of the arch having the abscissa, x.

M_d = the bending moment caused by statically determinate forces.

M_i = the bending moment caused by statically indeterminate forces.

$M_{d\text{-}LL}$, $M_{d\text{-}DL}$, $M_{i\text{-}LL}$, $M_{i\text{-}DL}$ = respectively, the statically determinate moments due to live load and dead load on a structure, and the statically indeterminate moments due to live load and dead load on the structure.

N = the normal component of the resultant thrust acting on a radial section of the arch.

N_x, T_x, S_x = respectively, the normal thrust, resultant thrust, and shear at a section of the arch having the abscissa, x.

p = an exponent of stress determined from experiment and defining the rate of plastic deformation with a change in stress. The number, p, may also appear as a coefficient in equations which have been differentiated or integrated.

q = an exponent of time determined from experiment and defining the rate of plastic deformation with duration of loading.

$P_1, P_2, \ldots P_n$ = concentrated loads.

$$Q = \left(b\, c^{\frac{1+2p}{p}} \times \frac{2p}{1+2p} \right).$$

$$S = \left(b\, c^{\frac{1+2m}{m}} \times \frac{2m}{1+2p} \right).$$

S = also, the shear component of the resultant thrust acting on a radial section of the arch.

s = the length of the arch axis.

T = the resultant thrust on a radial section of the arch.

t = the duration of a stressed condition, in days.

T_{RS}, N_{RS}, S_{RS}, and M_{RS} = the resultant thrust, normal thrust, shear, and moment at a section in an arch due to a horizontal rib-shortening thrust, $H_{0\text{-}RS}$.

$$U = b\, c^{\frac{1+m}{m}} \times \frac{m}{1+m} \left[\left(\frac{u'}{c} + 1 \right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1 \right)^{\frac{1+m}{m}} \right].$$

$$U' = \frac{m}{1+m} \left[\left(\frac{u'}{c} + 1 \right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1 \right)^{\frac{1+m}{m}} \right].$$

$$V = b\, c^{\frac{1+p}{p}} \times \frac{p}{1+p} \left[\left(\frac{v'}{c} + 1 \right)^{\frac{1+p}{p}} - \left(\frac{v'}{c} - 1 \right)^{\frac{1+p}{p}} \right].$$

$$V' = \frac{p}{1+p} \left[\left(\frac{v'}{c} + 1 \right)^{\frac{1+p}{p}} - \left(\frac{v'}{c} - 1 \right)^{\frac{1+p}{p}} \right].$$

V_0 = the vertical reaction at the center of co-ordinates, considered to act at the end of a rigid arm fixed to the arch at the springing.

u = the distance from the center of elastic deformation to a fiber in a section.

u' = the distance from the center of elastic deformation at a section to the centroidal axis.

v = the distance from the center of plastic deformation to a fiber in a section.

v' = the distance from the center of plastic deformation at a section to the centroidal axis.

$V_1, V_2, \ldots V_n$, and $H_1, H_2, \ldots H_n$ = the vertical and horizontal or quasi-horizontal components, respectively, of the forces, $P_1, P_2, \ldots P_n$.

V_R, V_L = the vertical reaction at the right and left supports, respectively, of an arch considered as a simple beam.

w = the magnitude of a uniformly distributed load per foot of length. (In a few instances, w is used to designate the distance from the center of combined elastic and plastic deformation to a fiber in a section.)

x, y = respectively, abscissa and ordinate of a point referred to a system of co-ordinates.

z = the distance of a fiber from the centroidal axis.

z' = the eccentricity of the line of thrust with respect to the arch axis.

I.—GENERAL STATIC AND GEOMETRIC RELATIONS IN A DEFORMED ARCH

Fundamental Relations.—In considering the action of any structure under load, be the structure statically determinate or indeterminate, attention is given to three principal items. These are:

1.—Geometric Relations.—The relation of the angle changes and translations of the differential parts of the structure must be such that the geometric requirements imposed on the structure are fulfilled, for example, constant span length, etc.

2.—Static Equilibrium.—The sum of the components in any direction of all forces acting on the whole structure, or on any part of the structure, must be equal to zero; the sum of the moments acting on the whole, or on any part of, the structure must be equal to zero.

3.—Properties of the Material of Construction.—Deformations and stresses in differential parts of the structure must be so related as to satisfy the properties of the material, for example, the ratio of stress to strain should correspond to that obtained by experiment.

If a structure is statically determinate, the reactions resulting from a given condition of loading can be found without considering the properties of the material. However, even then the internal stresses due to a given loading are dependent on the elastic properties of the material, and in some cases may involve the plastic properties or deformation with time as well as stress.

When a statically indeterminate structure is analyzed, all three of the aforementioned factors must be combined into equations before a satisfactory solution can be obtained.

Development of Geometric Relations.—Consider an arch of arbitrary shape (Fig. 1) subjected to any system of loading in its plane. Due to the assumed loads, P_1, P_2, P_n, there are the reactions, R_L and R_R, at the left and right ends respectively.

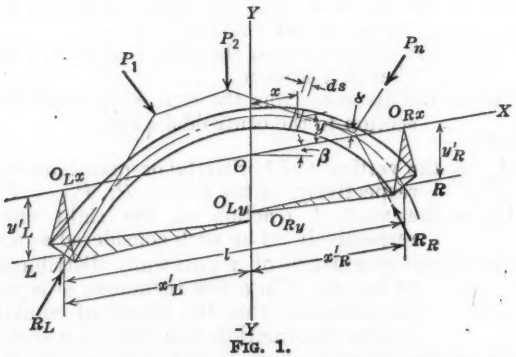

FIG. 1.

After loading (Fig. 2), the arch assumes a new conformation dependent on the movement of the abutments, the bending, and the elongation and shortening of elemental lengths of the arch rib. The deformations may be due to any one or any combination of the following causes:

(1) Rotation or translation of the abutments.
(2) Temperature change.
(3) Shrinkage of materials composing the arch.
(4) Plastic flow of materials composing the arch.
(5) Elastic deformation due to stress.

Assume (Fig. 1) a center of co-ordinates, O, and co-ordinate axes, x and y. A rigid lever arm is assumed to be connected to each springing and extended

to the x-axis and parallel to the y-axis. The length of the left lever arm is y'_L; that at the right is y'_R. Similarly, rigid lever arms of lengths, x'_L and x'_R, are connected to the left and right springing, respectively, and extended to the y-axis and parallel to the x-axis.

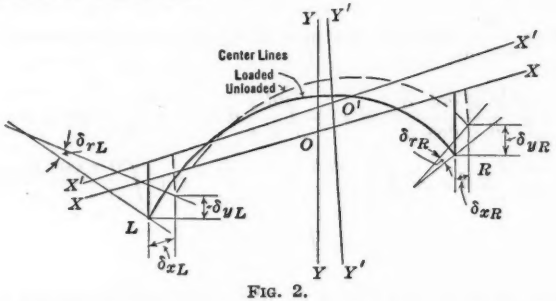

FIG. 2.

Consider a differential length of the arch rib, ds, the abscissa of which is x. The angle change within this differential length due to the strained condition of the arch, is $d\phi$. The rate of angle change then is given by $\frac{d\phi}{ds}$.

The relative movement of O_{Lx} and O_{Rx} in the direction of the x-axis, due to the differential angle change, $\frac{d\phi}{ds}$, is equal to $\frac{d\phi}{ds} y \cos\beta\, ds$.

Denote the shortening of a differential length, ds, the abscissa of which is x, by $d\xi$. Hence, the shortening (or lengthening) of the differential length in the x-direction will be $\frac{d\xi}{ds} \cos\alpha\, ds$, in which, α is the angle the arch axis makes with the x-axis.

Considering the sliding or shear deformation perpendicular to the arch axis in a length, ds, to be $d\tau$, the differential movement in the x-direction is found to be $\frac{d\tau}{ds} \sin\alpha\, ds$.

As a result of these considerations, the equation for the relative movement of O_{Lx} and O_{Rx} along the x-axis is,

$$\Delta l = \delta_{xR} - \delta_{xL} = \int \frac{d\phi}{ds} y \cos\beta\, ds + \int \frac{d\xi}{ds} \cos\alpha\, ds + \int \frac{d\tau}{ds} \sin\alpha\, ds - (y'_L\, \delta_{rL} - y'_R\, \delta_{rR}) \cos\beta \quad \ldots\ldots\ldots\ldots (1)$$

Similarly, it may be shown that the relative movement of O_{Ly} with respect to O_{Ry} along the y-axis is,

$$\Delta l' = \delta_{yR} - \delta_{yL} = \int \frac{d\phi}{ds} x \cos\beta\, ds + \int \frac{d\xi}{ds} \sin(\alpha - \beta)\, ds + \int \frac{d\tau}{ds} \cos(\alpha - \beta)\, ds - (x'_L\, \delta_{rL} - x'_R\, \delta_{rR}) \cos\beta \quad \ldots\ldots\ldots (2)$$

The rotation of the left springing with respect to the right springing is,

$$\Delta r = \delta_{rR} - \delta_{rL} = \int \frac{d\phi}{ds} ds \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Equations (1), (2), and (3) define the changed configurations of the arch due to all possible causes. These relations are geometrical and are independent of the properties of the material or conditions of equilibrium.

Considering that there is no translation or rotation of the ends of the arch, and substituting $\frac{dx}{\cos \alpha}$ for $d\,s$, the preceding geometric relations reduce to the forms,

$$\left.\begin{aligned} 0 &= \int \frac{d\,\phi}{d\,s}\frac{\cos\beta}{\cos\alpha}\,y\,d\,x + \int \frac{d\,\xi}{d\,s}\,d\,x + \int \frac{d\,\tau}{d\,s}\tan\alpha\,d\,x \\ 0 &= \int \frac{d\,\phi}{d\,s}\frac{\cos\beta}{\cos\alpha}\,x\,d\,x + \int \frac{d\,\xi}{d\,s}\frac{\sin(\alpha-\beta)}{\cos\alpha}\,d\,x + \int \frac{d\,\tau}{d\,s}\frac{\cos(\alpha-\beta)}{\cos\alpha}\,d\,x \\ 0 &= \int \frac{d\,\phi}{d\,s}\frac{d\,x}{\cos\alpha} \end{aligned}\right\} \quad (4)$$

The changes in conformation of an unsymmetrical fix-ended arch subjected to an arbitrarily chosen condition of loads, temperature change, shrinkage, etc., are thus defined by Equations (4).

For a symmetrical arch these equations may be written in the form:

$$\left.\begin{aligned} 0 &= \int \frac{d\,\phi}{d\,s}\frac{y}{\cos\alpha}\frac{d\,x}{} + \int \frac{d\,\xi}{d\,s}\,d\,x + \int \frac{d\,\tau}{d\,s}\tan\alpha\,d\,s \\ 0 &= \int \frac{d\,\phi}{d\,s}\frac{x}{\cos\alpha}\frac{d\,x}{} + \int \frac{d\,\xi}{d\,s}\tan\alpha\,d\,x + \int \frac{d\,\tau}{d\,s}\,d\,x \\ 0 &= \int \frac{d\,\phi}{d\,s}\frac{d\,x}{\cos\alpha} \end{aligned}\right\} \quad (5)$$

Relations Defining Static Equilibrium.—An unsymmetrical arch (Fig. 3) is subjected to an arbitrary system of loading, P_1, P_2, P_n, in the plane of the arch, with reactions, R_L and R_R, occurring at the left and right ends of the arch, respectively. Fig. 3 shows the arch under load and in its deformed state.

The arch has been made statically determinate with respect to the external forces and reactions in Fig. 4, while at the same time it is held in its deformed state by forces applied to the ends of rigid lever arms which are fixed to the left and right springings respectively, and terminate at the center of coordinates. The arch in its deformed state is held in equilibrium by a system of forces which may be divided into two classes, namely, (1) statically determinate forces and reactions, and (2) redundant forces.

The statically determinate forces and reactions are, respectively, the applied loads, P_1, P_2, P_n, and the reactions, V_L, V_R, and H_R. The reaction, H_R, is a quasi-horizontal force having its point of application at the right end of the arch and its direction parallel to the x-axis. The statically determinate forces may be considered as an independent system holding the structure in equilibrium. These are independent of internal stresses in the arch caused by deformations. Thus,

$$\Sigma\, H_R = \Sigma\, H_n$$
$$V_L + V_R = \Sigma\, V_n$$
$$\Sigma\, M \text{ (due to } P) = \Sigma\, M \text{ (due to reactions)}$$

On the other hand, the redundant forces are caused by restrained conditions of the ends of the arch and are directly caused by temperature changes, rib-shortening, movement of abutments, etc. The components of these forces must exist in pairs equal in magnitude and opposite in sense, for unless this is true the structure will not be maintained in a state of equilibrium. The redundant forces required to hold the structure in the conformation assumed are arbitrarily divided into H_0, V_0, and M_0-components at the center of the co-ordinate system. The H_0-components act along the x-axis, whereas the V_0-components are directed along the y-axis. These components may be considered to act in pairs of equal and opposite forces, one set acting upon the end of each lever arm.

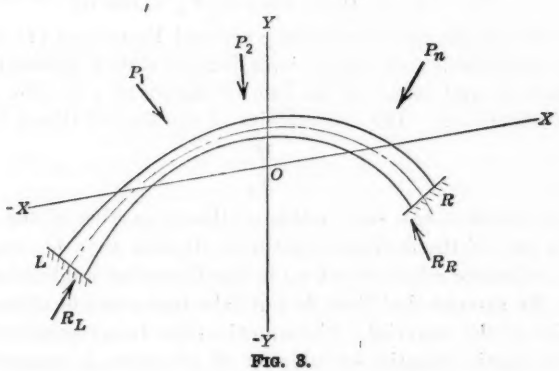

FIG. 3.

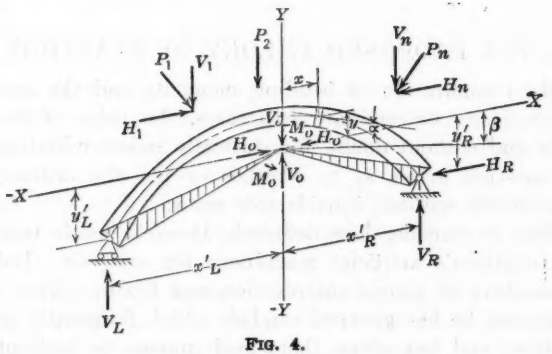

FIG. 4.

It is well to remember, comparing Figs. 3 and 4, that the resultant of the components, H_0, V_0, and M_0, at the end of the left lever arm, combined with the vertical reaction, V_L, is the reaction, R_L. Similarly, the resultant of the components considered to act at the end of the right lever arm and the reactions, V_R and H_R, form the reaction, R_R.

Consider, now, a normal section through the arch axis, having the abscissa, x. The stress functions due to the statically determinate forces will be denoted by a subscript, d; functions due to redundant forces will be denoted by a subscript, i. Hence, the moment at a section of the arch having the abscissa, x,

will be given by $M_x = M_{xd} + M_{xi}$. Thus, considering the sense of the redundant forces on the left lever arm, the moment, normal thrust, and shear, respectively, are found to be:

$$\left.\begin{aligned} M_x &= M_{xd} + M_0 + V_0\, x \times \cos\beta - H_0\, y \times \cos\beta \\ N_x &= N_{xd} + H_0 \times \cos\alpha + V_0 \times \sin(\alpha - \beta) \\ S_x &= S_{xd} + H_0 \times \sin\alpha + V_0 \times \cos(\alpha - \beta) \end{aligned}\right\} \quad \ldots\ldots\ldots (6)$$

For the case of a symmetrical arch referred to a rectangular system of co-ordinates, Equations (6) become:

$$\left.\begin{aligned} M_x &= M_{xd} + M_0 + V_0\, x - H_0\, y \\ N_x &= N_{xd} + H_0 \times \cos\alpha + V_0 \times \sin\alpha \\ S_x &= S_{xd} + H_0 \times \sin\alpha + V_0 \times \cos\alpha \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (7)$$

Equations (6) for the unsymmetrical arch and Equations (7) for the symmetrical arch, respectively, are simply equations of statics defining the magnitude, line of action, and sense of the line of thrust at a section of the arch the abscissa of which is x. The eccentricity of the line of thrust is,

$$z' = \frac{M_x}{N_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Thus far, no account has been taken of the properties of the material of construction in any of the derived equations. It may then be concluded that the geometric and static relations set up in the foregoing analysis are not open to question on the ground that they do not take into consideration the correct elastic properties of the material. The equations are true regardless of whether the material is elastic, plastic, or whether it possesses a composite of both properties.

II.—THE PROPOSED THEORY OF ELASTICITY

While in the computation of bending moments and the action of outer forces on all statically determinate structures, the rules of statics may be applied directly and without doubt, for statically indeterminate structures of concrete, the question arises as to whether or not the ordinary theory of elasticity is applicable without considerable error.

It is appalling to consider how definitely Hooke's law is ingrained in all the structural engineer's artificial machinery for analysis. Indeed, he has started with members in simple compression and flexure; from their mathematical development he has grouped symbols which frequently appear in the same combination, and has given them such names as moment of inertia, elastic weight, etc. Not satisfied with these, he has further superimposed laws governing the relation of these symbols to each other, and has evolved such combinations as Maxwell's law of reciprocal deflections, the theorems of virtual work, methods of substitute beams, moment areas, slope deflections, and others. All these methods and theorems are developed assuming the variation of stress with strain to be governed by Hooke's law.

The assumption of a constant elastic modulus is correct for steel within the elastic limit, at least as far as designs subjected only to static loads are concerned. However, for concrete, this assumption is known to be incorrect and has resulted in much controversy. Many have doubted the applicability

of the ordinary theory of elasticity to indeterminate concrete structures. To be sure, experiments are recorded indicating that the theory of elasticity gives results at least approximately correct for short-time loads. From a group of experiments performed on concrete frames,* M. Abe, M. Am. Soc. C. E., concludes that, "the elastic action of the frames under external load and the manner of stress distribution along members of the frame agree fairly well with the analysis given." (Dr. Abe's analysis was an application of a form of the ordinary theory of elasticity.†) Theoretical studies of the "Austrian Experiments on Arches"‡ indicates that the theory of elasticity, at least for loads within the working stresses of the material, gives reasonable results in a stress analysis for rubble masonry and brick arches as well as for concrete arches.

However, considering the degree of precision obtained by the ordinary theory of elasticity, it is unscientific, to say the least, not to determine what effect a variable modulus of elasticity has upon the moment and stress distribution within a structure. Of course, a theory based merely upon the properties of the material and not yet substantiated by at least a few scientific tests upon simple structures is not entirely ready for complete acceptance. Yet, no investigations are so unscientific and unreliable as those made upon built-up units, before a feasible theory of analysis has been developed. It is these considerations that have led the writer to develop what he believes to be a more flexible theory of elasticity, that is, one which accounts for a variable modulus of elasticity.

It is of some interest to note that Professor Melan (in studies to determine the modulus of elasticity of the concrete forming the "Stampf-Beton" arches tested by the Austrian Committee) found that the computed elastic modulus changed very greatly as the load increased, being less for the greater loads. His work was carried out by taking the actual vertical and horizontal displacements into account in an ordinary elastic analysis. Thus, one might conclude that the study of deformations in themselves have little meaning in the investigation of stresses in arches.

Basis for Proposed Theory.—After studying the stress-strain relation of considerable experimental data for concrete, the writer has chosen the equation,

$$\varepsilon = k\,\sigma^m$$

to define the relation. Bach§ developed an equation similar to this except that instead of k he used the reciprocal of the initial modulus of elasticity. However, this expression did not prove as satisfactory to the writer and is less flexible in application. Bach and Schüle‖ through a large number of experiments found the exponent, m, in their equation to range between the limits of 1.10 and 1.16 for various mixes of concrete.

For concrete which was made into tension and compression specimens at the same time it was found that the elastic modulus in tension¶ did not differ

* *Bulletin No. 107*, Univ. of Illinois Eng. Experiment Station, p. 100.

† Assuming stress proportional to strain and no plastic deformations.

‡ Bericht des Gewölbe-Ausschusses, *Zeitschrift* des Oesterr. Ingenieur- und Architekten-Vereines, 1896.

§ Bach and Bauman, "Elastizität und Festigkeit", Berlin, 1924.

‖ *Zeitschrift* des Vereines deutscher Ingenieure, 1898.

¶ *Mitteilungen* über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens. Heft 227, pp. 49-51.

greatly from that in compression, loads being applied in steps for the first time. This relation does not necessarily follow in all cases, but seems to be true in general for loads applied for the first time.

Data obtained from typical experiments by Dr. E. Probst* have been used by the writer as a basis for analysis in the basic equation, and curves (not herein included) were found which fit the data exceedingly accurately. The data included were for concrete 70 days old and of a 1 : 2 : 4 mix made with 10.1% water, giving a "very plastic material." Using the English system of units, for compression, the data are defined by,

$$\varepsilon = 178 \times 10^{-9}\ \sigma^{1.105}$$

In tension the elasticity of the specimens used is represented by,

$$\varepsilon = 192 \times 10^{-9}\ \sigma^{1.03}$$

Development of Proposed Theory of Elasticity for Flexure.—When analyzing a concrete structure, it is hopeless to get a theory that is altogether exact. However, close approximations may be made, resulting in a study which will approach the actual conditions. Concrete in tension will follow a slightly different law for the relation of stress to deformation from that followed in compression. However, the general exponential relation is close for both. If the permanent set be included in the deformation represented in the exponential equation, then the equation, based on Probst's experiments, is practically the same for tension and compression.

Based upon these considerations, a theory of analysis is developed which accounts for a variation of the modulus of elasticity with variation in stress.

Structures are grouped into two classes, namely, (1) those subjected to pure flexure; and (2) those subjected to combined flexure and compression.

Development of the Flexure Equations According to the Proposed Theory of Elasticity.—The relation between elastic deformation and stress is given by,

$$\varepsilon = k\ \sigma^m \qquad (9)$$

Fig. 5 indicates a beam loaded in an arbitrary manner. Consider a differential length, $d\,s$. The section of the beam (Fig. 6(a)) is assumed rectangular, having a width, b, and depth, d, or $2c$.

Let, $A = b\,d$ = the area of the section. By the principles of statics,

$$M = \int \sigma_z\, z\, d\, A$$

or, considering σ_z to be independent of b,

$$M = \int_{-c}^{c} b\ \sigma_z\, z\, d\, z = 2\, b \int_{0}^{c} \sigma_z\, z\, d\, z \qquad (10)$$

A transformation of Equation (9) shows that,

$$\sigma_z = \left(\frac{\varepsilon_z}{k}\right)^{\frac{1}{m}} \qquad (11)$$

From Fig. 6(b), it may be seen that, assuming conservation of plane sections during bending,

$$\varepsilon_z = z\, \frac{d\ \phi_E}{d\ s} \qquad (12)$$

* "Einfluss der Armature und der Risse im Beton auf die Tragsicherheit," E. Probst, *Mitteilungen* aus dem Königlichen Materialprüfungsamt zu Gross-Lichterfelde West, Ergänzungsheft I, pp. 8-14, Verlg. von Julius Springer, 1907.

Combining Equations (10), (11), and (12), and rewriting,

$$M = \frac{2\ b}{k^{\frac{1}{m}}} \left(\frac{d\ \phi_E}{d\ s}\right)^{\frac{1}{m}} \int_0^c z^{\frac{1+m}{m}}\ d\ z$$

from which, when integrated and transformed,

$$\frac{d\ \phi_E}{d\ s} = \frac{k\ M^m}{\left[\frac{2\ m}{1+2\ m}\ b\ c^{\frac{1+2\ m}{m}}\right]^m}$$

or,

$$\frac{d\ \phi_E}{d\ s} = k \left(\frac{M}{Q}\right)^m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which,

$$Q = \frac{2\ m}{1+2\ m}\ b\ c^{\frac{1+2\ m}{m}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

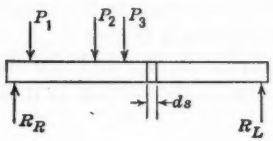

FIG. 5.

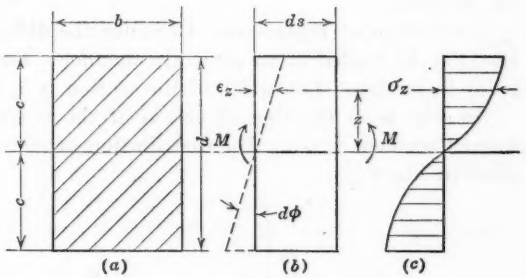

FIG. 6.

Equation (13) defines the rate of angle change at a section of a rectangular beam in terms of the bending moment, dimensions of the section, and elastic properties of the material.

Special Case in Which $m = 1$.—Assuming that $m = 1$, as in the ordinary theory of elasticity,

$$Q = \frac{2}{3}\ b\ c^3 = \frac{1}{12}\ b\ d^3 = I$$

and,

$$\frac{d\ \phi_E}{d\ s} = \frac{M}{E\ I}\ *$$

DERIVATION OF PROPOSED THEORY OF ELASTICITY FOR ARCHES

General Relations.—A method of analysis in which account is taken of the fact that strain is an exponential function of the stress becomes somewhat complicated because the law of superimposition of stresses and strains does not apply; that is, in order to reach the final stress and strain condition, one may not combine the results obtained by considering separately the effects of indi-

* The value, E, is here assumed to be equal to $\frac{1}{k}$. As a matter of fact, k should be chosen from experimental data and a value differing somewhat from this will better suit the data.

vidual components of a force system. For example, consider a member in simple flexure. The total bending moment, M, is equal to the sum of the bending moments due to the individual loads, P_1, P_2, ..., P_n, or $M = M_1 + M_2 .... + M_n$. The rate of angle change at any section is,

$$\frac{d\ \phi_E}{d\ s} = k \left(\frac{M}{Q}\right)^m = k\ \frac{(M_1 + M_2 + ... + M_n)^m}{Q^m}$$

This equation can not be simplified to a form,

$$\frac{d\ \phi_E}{d\ s} = \frac{M_1}{E\ I} + \frac{M_2}{E\ I} + ... + \frac{M_n}{E\ I}$$

as in the ordinary theory of elasticity. As a result, when dealing with statically indeterminate structures, it is necessary to handle the entire force system in a single analysis.

Since shear deformations have only slight effect on the change in conformation of a loaded arch rib in comparison to the effects of moment and rib-shortening deformations, the deformations due to shear will be neglected in the analysis.

Derivation of Equations.—Consider the differential length, $d\ s$, of an arch rib (Fig. 7) loaded in an arbitrary manner. The rib is assumed to be rectangular in section. The width of the section is b, and the depth is d or $2c$.

In Fig. 8(a) the line of action of the normal thrust is N_x, and the deformed segment is $d\ s$. The distribution of stresses across the section is indicated in Fig. 8(b).

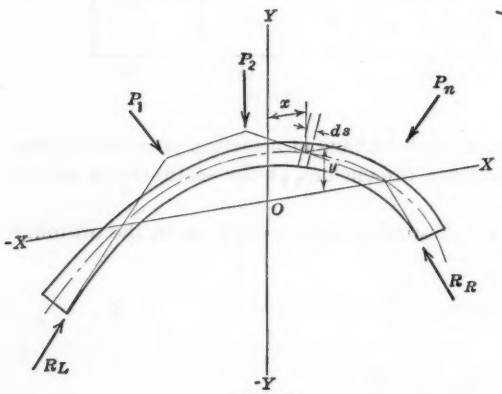

FIG. 7.

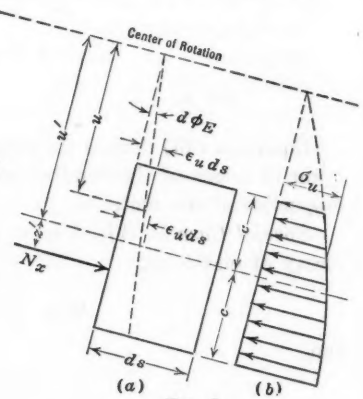

FIG. 8.

The relation of strain to stress is given by,

$$\varepsilon_u = k\ \sigma_u^m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

By geometry (Fig. 8(a)),

$$\varepsilon_u = u\ \frac{d\ \phi_E}{d\ s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Subtracting Equation (16) from Equation (15) and re-arranging terms,

$$\sigma_u = \left(\frac{u}{k} \times \frac{d\ \phi_E}{d\ s}\right)^{\frac{1}{m}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

The following relations must be true in order to maintain static equilibrium:

$$N_x\,(u' + z') = \int \sigma_u\, u\, d\, A \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

$$N_x = \int \sigma_u\, d\, A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

By substituting $b \times d\,u$ for $d\,A$, the differential area of the section, and the value found in Equation (17), for σ_u, Equation (19) becomes,

$$N_x\,(u' + z') = \frac{b}{k^{\frac{1}{m}}} \times \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \int_{(u'-c)}^{(u'+c)} u^{\frac{1+m}{m}}\, d\,u$$

from which,

$$N_x(u' + z') = \frac{m}{1+2m} \times \frac{b}{k^{\frac{1}{m}}} \times \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \left[(u' + c)^{\frac{1+2m}{m}} - (u' - c)^{\frac{1+2m}{m}}\right] . (20)$$

Similarly, by making substitutions for $d\,A$ and σ_u in Equation (19), and integrating between the same limits,

$$N_x = \frac{m}{1+m} \times \frac{b}{k^{\frac{1}{m}}} \times \left(\frac{d\,\phi_E}{d\,s}\right)^{\frac{1}{m}} \left[(u' + c)^{\frac{1+m}{m}} - (u' - c)^{\frac{1+m}{m}}\right] . . (21)$$

Equation (20) divided by Equation (21) gives,

$$(u' + z') = \frac{1+m}{1+2m} \times \frac{\left[(u' + c)^{\frac{1+2m}{m}} - (u' - c)^{\frac{1+2m}{m}}\right]}{\left[(u' + c)^{\frac{1+m}{m}} - (u' - c)^{\frac{1+m}{m}}\right]} \ldots\ldots . (22)$$

Transposing u' to the second member of the equation and dividing both members by c,

$$\frac{z'}{c} = \frac{1+m}{1+2m} \times \frac{\left[\left(\frac{u'}{c} + 1\right)^{\frac{1+2m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+2m}{m}}\right]}{\left[\left(\frac{u'}{c} + 1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+m}{m}}\right]} - \frac{u'}{c} \ldots (23)$$

From Equation (21),

$$\frac{d\,\phi_E}{d\,s} = k\, \frac{N_x^{\,m}}{b\, c^{\frac{1+m}{m}}\, \frac{m}{1+m} \left[\left(\frac{u'}{c} + 1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+m}{m}}\right]^m}$$

or,

$$\frac{d\,\phi_E}{d\,s} = k \left(\frac{N_x}{U}\right)^m \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which,

$$U = \left\{ b\, c^{\frac{(1+m)}{m}}\, \frac{m}{1+m} \left[\left(\frac{u'}{c} + 1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c} - 1\right)^{\frac{1+m}{m}}\right] \right\} . . (25)$$

The rate of angle change in the arch at a radial section the abscissa of which is x, is given by Equation (24) in terms of the dimensions of the section, elastic properties of the material, magnitude of the normal thrust, and distance between the center of rotation and center of gravity of the section. Equation (23) shows the relation between the eccentricity of the thrust and the distance to the center of rotation. Thus, Equations (23) and (24) together

fully define the angle change in terms of the moment, thrust, and properties of the section.

The rib-shortening, or the shortening at the gravity axis, of an elemental length, $d\,s$, is,

$$d\,\xi_E = \varepsilon_{u'}\,d\,s$$

but by geometry,

$$\varepsilon_{u'} = u'\,\frac{d\,\phi_E}{d\,s}$$

hence,

$$\frac{d\,\xi_E}{d\,s} = u'\,\frac{d\,\phi_E}{d\,s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Substituting for $\frac{d\,\phi_E}{d\,s}$, the value found in Equation (24),

$$\frac{d\,\xi_E}{d\,s} = u'\,k\left(\frac{N_x}{U}\right)^m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Neglecting shear deformations and substituting values for $\frac{d\,\phi}{d\,s}$ and $\frac{d\,\xi}{d\,s}$, respectively, from Equations (24) and (27) into the geometric Equations (5),

$$\left.\begin{aligned} 0 &= \int k\left(\frac{N}{U}\right)^m \frac{y\,d\,x}{\cos\alpha} + \int u'\,k\left(\frac{N}{U}\right)^m d\,x \\ 0 &= \int k\left(\frac{N}{U}\right)^m \frac{x\,d\,x}{\cos\alpha} + \int u'\,k\left(\frac{N}{U}\right)^m \tan\alpha\,d\,x \\ 0 &= \int k\left(\frac{N}{U}\right)^m \frac{d\,x}{\cos\alpha} \end{aligned}\right\} \quad \ldots\ldots\ldots (28)$$

By substituting $\frac{M}{z'}$ for N in Equations (28), and replacing the definite integrals by summation of finite increments, the equations become,

$$\left.\begin{aligned} 0 &= k\sum\left(\frac{M}{z'\,U}\right)^m \frac{y\,\Delta\,x}{\cos\alpha} + k\sum u'\left(\frac{M}{z'\,U}\right)^m \Delta\,x \\ 0 &= k\sum\left(\frac{M}{z'\,U}\right)^m \frac{x\,\Delta\,x}{\cos\alpha} + k\sum u'\left(\frac{M}{z'\,U}\right)^m \tan\alpha\,\Delta\,x \\ 0 &= k\sum\left(\frac{M}{z'\,U}\right)^m \frac{\Delta\,x}{\cos\alpha} \end{aligned}\right\} \quad \ldots\ldots (29)$$

Special Case, $m = 1$.—Equations (29) defining the proposed theory of elasticity for arches readily reduce to those of the ordinary theory of elasticity.

Solution of Proposed Elastic Equations.—Equations (29) fully define the elastic equilibrium of the arch, that is, its change in shape. However, a solution of these equations is not as simple as that when the ordinary straight-line, stress-strain variation is assumed. Angle changes in elementary radial segments of the arch are dependent not only on the bending moment, but also on the eccentricity of the force producing the moment. No particular choice of the center of co-ordinates seems to aid materially in the analysis, because the deformations due to separate forces acting independently may not be

superimposed to obtain the deformation caused by the combined system of forces.

Assume, now, the value of m to be known from experiment. From Equation (23), values may then be computed showing the relation between the eccentricity, z', of the compressive force, and the distance, u', between the center of rotation and the gravity axis of the section. A more general relation is obtained by finding the relation between $\frac{z'}{c}$ and $\frac{u'}{c}$, because such a relation is directly applicable to a section of any thickness. Using constant values of m, curves have been plotted (Fig. 9) with $\frac{z'}{c}$ values as abscissas and $\frac{u'}{c}$ values as ordinates.

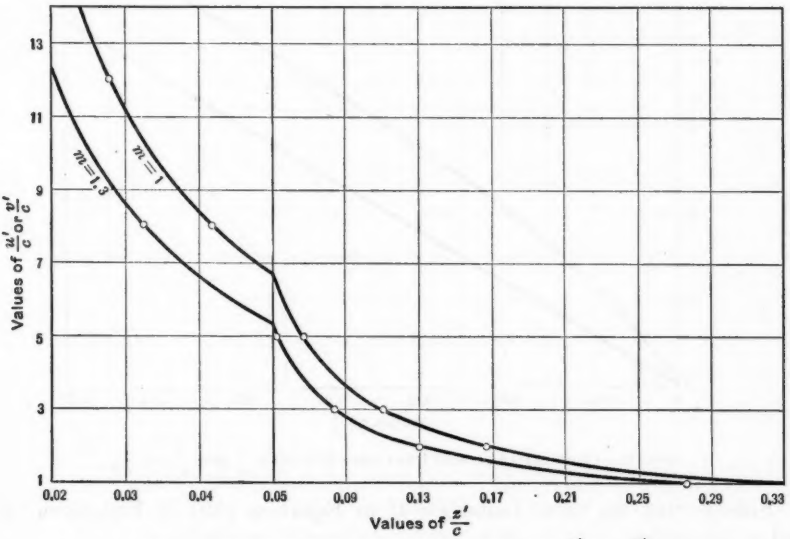

FIG. 9.—CURVES EXPRESSING RELATION BETWEEN $\frac{y'}{c}$ and $\frac{v'}{c}$.

From Equation (25),

$$U = b\, c^{\frac{1+m}{m}}\, U' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which,

$$U' = \frac{m}{1+m}\left[\left(\frac{u'}{c}+1\right)^{\frac{1+m}{m}} - \left(\frac{u'}{c}-1\right)^{\frac{1+m}{m}}\right] \ldots\ldots\ldots (31)$$

A relation of $\frac{1}{U'}$ and $\frac{z'}{c}$ is obtained between Equations (23) and (31) by substituting the same value of $\frac{u'}{c}$ in both equations and solving, respectively, for the corresponding values of $\frac{1}{U'}$ and $\frac{z'}{c}$. Curves have been plotted (Fig. 10) with

$\frac{z'}{c}$ values as abscissas and $\frac{1}{U'}$ values as ordinates. For $m = 1$, the curve is a straight line passing through the origin of co-ordinates. Hence, the expression, $\frac{1}{\frac{z'}{c} U'}$, is constant for all values of z'. Consequently, Equations (32), which have been derived from Equations (29), take the form of the ordinary theory of elasticity for $m = 1$.

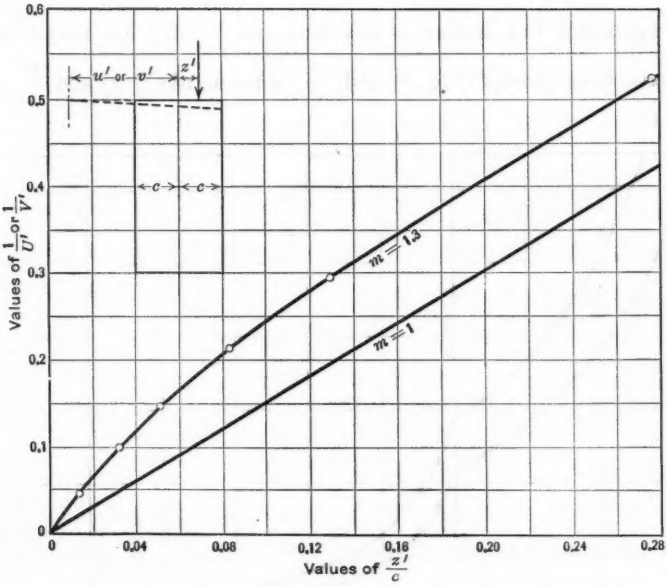

FIG. 10.—CURVES EXPRESSING RELATIONS BETWEEN $\frac{z}{c}$ and $\frac{1}{u'}$ or $\frac{1}{v'}$.

Substituting the value found for U in Equation (30) in Equations (29), and re-arranging,

$$\left.\begin{aligned} 0 &= k\left[\sum \frac{M_x^m}{\left(\frac{z'}{c}U'\right)^m}\frac{y\,\Delta x}{b^m c^{1+2m}\cos\alpha} + \sum \frac{u'}{c}\frac{M_x^m}{\left(\frac{z'}{c}U'\right)^m}\frac{\Delta x}{b^m c^{2m}}\right] \\ 0 &= k\left[\sum \frac{M_x^m}{\left(\frac{z'}{c}U'\right)^m}\frac{x\,\Delta x}{b^m c^{1+2m}\cos\alpha} + \sum \frac{u'}{c}\frac{M_x^m}{\left(\frac{z'}{c}U'\right)^m}\frac{\tan\alpha\,\Delta x}{b^m c^{2m}}\right] \\ 0 &= k\sum \frac{M_x^m}{\left(\frac{z'}{c}U'\right)^m}\frac{\Delta x}{b^m c^{1+2m}\cos\alpha} \end{aligned}\right\} \quad (32)$$

The factor, k, may be dropped, of course, from all these equations unless some element, such as shrinkage or temperature change, is taken into account.

Analyses of arches under various particular conditions of loading and deformation have been carried out by the proposed theory of elasticity, using Equations (32) and Figs. 9 and 10.

Importance of Variable Modulus of Elasticity.—The complexity of the angle changes in an arch is probably best realized by a few preliminary investigations. Upon first thought, one might conclude that the secant modulus for the average working stress can not be far wrong. However, the combined action of compression and bending moment introduces complications which are probably not immediately anticipated.

As an example, consider Fig. 11. The curve indicates the differential angle change at a given section due to a constant bending moment, the only variation being the eccentricity of the force producing the moment. It has been plotted on the basis of the exponential equation for the relation of stress to strain, considering $m = 1.3$. The ordinates are plotted as a constant, f, times $\frac{d\phi}{ds}$. The values of points on the curve are readily found by the aid of Fig. 10. If a constant secant modulus were adopted, and Hooke's law assumed to apply, the curve would take the form of a vertical straight line.

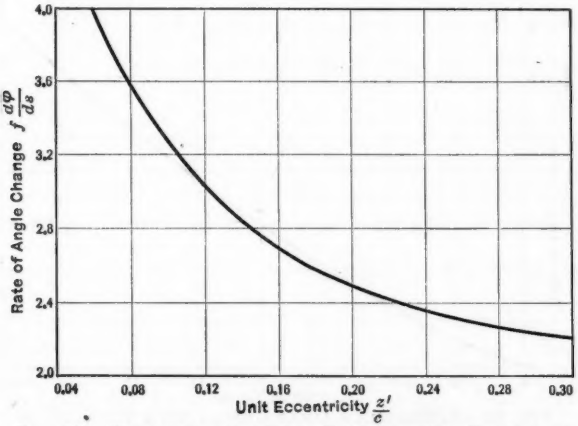

FIG. 11.—VARIATION OF ANGLE CHANGE WITH CHANGE IN ECCENTRICITY OF FORCE. MOMENT CONSIDERED CONSTANT. $m = 1.3$.

These considerations would indicate that no satisfactory study of the effect of a variable modulus of elasticity on the action of arches can be made, unless the eccentricity of the line of thrust as well as the bending moment is included in the analysis.

Consider a simple example solved by means of Fig. 11. No particular units will be kept in mind. Assume a load of 20 applied on a specimen, at an eccentricity, $0.06\frac{z'}{c}$. When $m = 1.3$, the rotation will be,

$$\phi = (20 \times 0.06)^{1.3} \times 3.95 = 5.00$$

Assume, now, that the same rotation is obtained, in a specimen of the same shape but having $m = 1$ as an index of the stress-strain relation, for a load of 20 placed at the same eccentricity.

Now, consider the loads removed, and a load of 3 placed at an eccentricity, $0.26\frac{z'}{c}$, on both specimens. The rotation will be:

For $m = 1$,

$$\phi = 5.00 \times \frac{3 \times 0.26}{20 \times 0.06} = 3.25$$

and for $m = 1.3$,

$$\phi = (3 \times 0.26)^{1.3} \times 2.3 = 1.66$$

Hence, while the rotation in both specimens may be the same for one condition of loading, it may be quite different for another condition even though there is little change in bending moment. This analysis corresponds to taking a secant modulus for one case, that is, when $m = 1$, and the exponential formula in the other case.

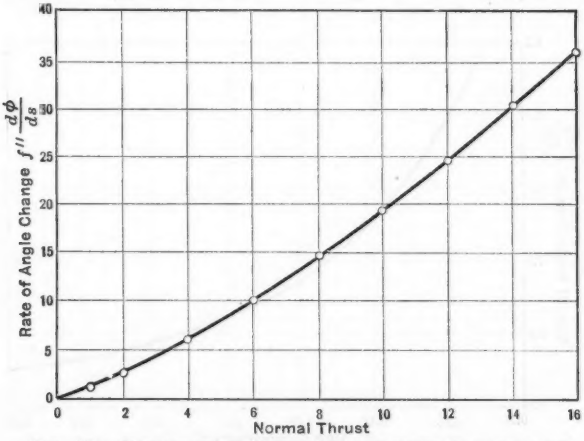

FIG. 12.—VARIATION OF ANGLE CHANGE WITH VARIATION OF INTENSITY OF NORMAL THRUST. ECCENTRICITY OF LINE OF THRUST CONSIDERED CONSTANT, $m = 1.3$.

A further investigation has been plotted in Fig. 12. The eccentricity of the line of thrust through a particular section was assumed to remain the same, while the normal thrust was considered to vary. The value of m in the exponential formula was assumed to be equal to 1.3. The rate of angle change is plotted in terms of $\frac{d\phi}{ds}$ times a constant, f''.

Again, Fig. 13 shows the results of a study of the variation of angle change with the eccentricity of the line of thrust, considering the normal thrust to remain constant. The full line is drawn through points computed with the aid of Fig. 10; the broken line is a straight line drawn through the origin of co-ordinates at a slope arbitrarily chosen to fit closely the computed curve. The rate of angle change is plotted as $\frac{d\phi}{ds}$ times a constant, f'.

A comparison of Figs. 11, 12, and 13 shows how the action of a structure may be anticipated. Fig. 12 indicates that the rate of angle change at a section increases more rapidly than the thrust, while Fig. 11 shows that, for a given moment, the rate of angle change decreases with an increase in eccentricity of the thrust producing the moment.

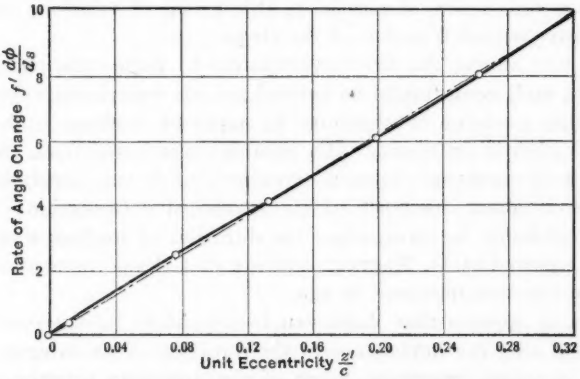

FIG. 13.—VARIATION OF ANGLE CHANGE WITH ECCENTRICITY OF LINE OF THRUST. NORMAL THRUST CONSIDERED CONSTANT. $m = 1.3$.

If the thrust remains constant (Fig. 13), the angle change varies almost directly as the eccentricity; that is, the curve representing the relation between the rate of angle change and the eccentricity is very nearly a straight line. Hence, considering an arch so proportioned that for a given loading the average stress, $\frac{N}{A}$, is the same on every normal section, and, neglecting rib-shortening effects, it might be expected that the ordinary theory of elasticity would give the location of the line of thrust very closely. However, if the average normal stress varies considerably from section to section, the line of thrust determined by the ordinary theory of elasticity could not be expected to be quite as accurate.

As a matter of fact the average compressive stress on the various normal sections throughout an arch of usual design does not vary to a large degree. For a uniformly loaded symmetrical parabolic arch, in which the sectional area increases as the secant of the angle between a tangent to the rib axis and the horizontal, the average compressive stress would be the same for every normal section.

III.—PROPOSED THEORY OF PLASTICITY

That property of concrete by which the material undergoes a gradual but persistent deformation while in the condition of sustained stress, has been the subject of considerable discussion during the last decade. This phenomenon of plasticity, often referred to as "flow of concrete" or "time-yield", has been recognized by the early concrete experimenters, but only comparatively recently has it been given special attention in investigations.

In the proposed theory of elasticity the total strain is an exponential function of the stress and independent of the time interval of the stress. In contradistinction that group of mathematical relations which treats of the deformations in a structure where the total strain is a function of the duration as well as intensity of stress, has been arbitrarily termed the "theory of plasticity". As in the proposed theory of elasticity, this group of relations accounts for strain as an exponential function of the stress.

C. Bach* was among the first investigators to notice the phenomenon of permanent set, and, accordingly, he carried out his experiments for the determination of the modulus of elasticity by repeated loading, each successive loading being slightly increased. His modulus was based upon the rebound.

As a result of numerous German experiments,† it was concluded that the permanent set is about one-tenth of the combined deformation. The latter conclusion is probably in error, since the duration of loading was not great. Experiments reported by G. Magnel‡ indicate that the permanent set becomes smaller as the concrete increases in age.

In general, it appears that American investigators in concrete have been foremost in realizing the importance of the duration of stress upon the deformations in a concrete structure. Some of the American experiments will be discussed in detail.

Proposed Exponential Equation of Plastic Deformation.—Investigations in the plasticity of concrete, for the most part, have been of a qualitative, rather than quantitative, nature. The ultimate test of all experimental data on a material of engineering construction is the adaptability of such data to transformation into true mathematical laws which will define the properties of the material in such a manner that they may be accounted for in the analysis of complete structures.

Study of test data indicates that, while a mathematical law defining the plasticity of concrete must be expressed in three variables, it is closely given by an exponential equation involving the three variables, two of which are independent.

Investigations of a qualitative nature have been published by F. R. McMillan, A. H. Fuller, and H. F. Clemmer, Members, Am. Soc. C. E., and S. C. Hollister, Assoc. M. Am. Soc. C. E., as well as by Messrs. M. B. Lagaard, E. B. Smith, C. O. More, and others. The data made available by these experiments have been used by the writer as a comparison to the exponential equation,

$$\rho = K \sigma^p t^q$$

in which, ρ is the plastic deformation which is a function of the unit stress and the time, t. The factor, K, is a parameter depending on the properties of the material. The exponents, p and q, are constants which depend on the properties of the material at the beginning of the stressed condition.

In proposing this equation, it is realized that the elastic and plastic properties of concrete vary considerably with the age of the material, especially

* *Zeitschrift* des Vereines deutscher Ingenieure, 1895, p. 489.

† *Deutscher Ausschuss für Eisenbeton,* Heft 17 (1912), pp. 66-67.

‡ "Recherches experimentales sur le coefficient d'élasticité du béton," G. Magnel, *Revue Universelle des Mines,* Vol. 67, No. 1, 1924.

during the first two or three months after it has been mixed. The values, K, p, and q, are intended to define the law of time-stress yield of the concrete during any time interval, provided the parameters satisfy the properties of the material at the beginning of the interval. The variable, t, is measured with its zero value at the instant that the load producing a stress, σ, is applied.

It will be observed that p and q are independent of the units of measurement of the stress, time, or deformation. Only K will be affected by a change in the units of measure.

Experimental Basis for Proposed Exponential Equation of Plastic Deformation.—Assuming that the plasticity of concrete may be expressed in terms of an exponential equation in which the time and stress intensity are independent variables, experimental data have been plotted on logarithmic paper to find the variable relation between deformation and time, for a constant stress. Similarly, experimental values have been plotted to determine the relation between deformation and stress for constant time intervals.

Many of the test data available have not been compiled in such a way that the effects of shrinkage and temperature deformations can be separated from the plastic deformations. However, these effects in the data considered are believed to be small enough not to affect the conclusions. No attempt has been made to convert all data examined to the same units, because only K is affected.

Time tests by Fuller and More* were performed on specimens presumably of approximately the same quality of concrete. The data have been plotted in Fig. 14. Specimens A and B were cylinders 8 in. in diameter, made from concrete mixed for use in building construction. Specimen C was a block 8 in. square and 13 in. high, cut from a floor of the same building. All specimens were subjected to different loadings before applying a constant load over a long period of time. Tests on Cylinder A were begun 101 days after pouring, and the time-yield test under a stress of 650 lb. per sq. in. was begun 24 days later. Cylinder B was 137 days old when tests were begun; a constant load causing a stress of 1 150 lb. per sq. in. was applied after 6 days of variable load tests. Specimen C was 131 days old, and the constant-load time-yield test was begun after 11 days, with a stress of 650 lb. per sq. in. No corrections have been made for temperature and moisture variations or for deformation due to shrinkage. However, comparatively little shrinkage will occur after 4 months, which time was the approximate age of the specimen. The tests having been made under laboratory conditions, it is believed that temperature and moisture variations were too small to affect the results materially.

No data were reported from which a relation between the intensity of stress and rate of deformation could be computed.

The exponential relation for time-yield for these experiments becomes,

$$\rho = f(\sigma^p)\, t^{0.42}$$

In spite of the fact that all specimens were treated differently in the early stages of the load tests and one was loaded with a greater stress intensity than

* "Time Tests of Concrete", by Fuller and More, *Proceedings*, Am. Concrete Inst., Vol. 12, p. 302, 1916.

the others, all deformed approximately as the 0.42 power of the time. The theoretical curves shown in Fig. 14 fit the experimental points fairly closely.

Mr. Earl B. Smith made several tests* to determine the rate of yield of stressed concrete, part of which were on a reinforced concrete beam of 1:2:4 mix with gravel as the coarse aggregate. Reinforcement consisted of five ¼-in. rods. The beam was 5 in. wide, 8 in. deep, including 1-in. steel cover, and had a clear span of 10 ft. About two weeks after pouring, the beam was loaded at the center with a concentrated load of 1 225 lb. Deformations were measured at the top of the beam by means of a strain-gauge. Since, for loads considerably below the ultimate, the outer fiber stress in a concrete section of a beam is approximately proportional to the moment, the relation of deformation and stress was found approximately by comparing the moment with the deformation.

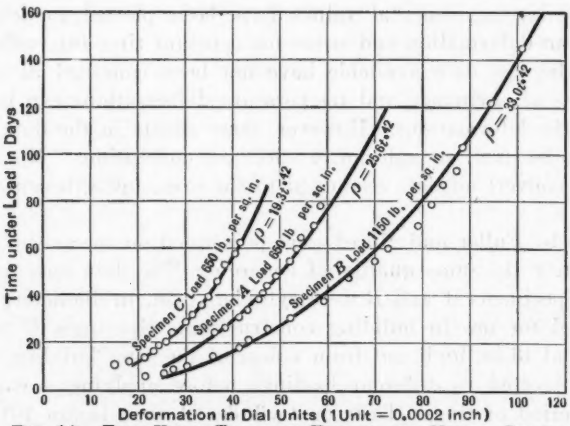
Time under Load in Days
Deformation in Dial Units (1 Unit = 0.0002 inch)
Specimen C, Load 650 lb. per sq. in.
Specimen A, Load 650 lb. per sq. in.
Specimen B, Load 1150 lb. per sq. in.
ρ = 19.3 t^0.42
ρ = 25.8 t^0.42
ρ = 33.0 t^0.42

FIG. 14.—TIME-YIELD TESTS BY FULLER AND MORE. CURVES BY PROPOSED EXPONENTIAL FORMULA.

It is realized, of course, that such measurements would be true only if the plastic deformation varies directly as the stress. Since the plastic deformation appears to vary more nearly as the 1.3 power of the stress, there is likely to be a redistribution of stress over the section as the period of loading continues, the stress decreasing near the outer fiber and increasing near the neutral axis. The neutral axis will move down slightly, because the concrete continues to deform while the steel remains unchanged for a given stress. Hence, some error is introduced by assuming the outer fiber stress constant during a time-yield test. However, this should not seriously affect the general law which is being derived subject to confirmation by more precise experiments.

As the beam was only two weeks old when the test was started, considerable deformation was due to shrinkage. Correction was made for this by subtracting the deformation over the support from the recorded plastic flow throughout the span, for each time interval.

The exponential equation, $\rho = K\,\sigma^p\,t^q$, for the concrete in Mr. Smith's test beam, takes the form,

$$\rho = 0.198 \cdot \sigma^{1.73}\, t^{0.18}$$

* "Flow of Concrete Under Sustained Loads", by E. B. Smith, *Proceedings*, Am. Concrete Inst., Vol. 13, p. 99, 1917; Vol. 12, p. 317, 1916.

in which, ρ is the deformation in 0.0001 in., measured over an 8-in. length; σ is the fiber stress measured in terms of the moment, in thousands of foot-pounds; and t is the time of the period of stress, in days. A change to any other units of measure will simply change the coefficient of the second member of the equation.

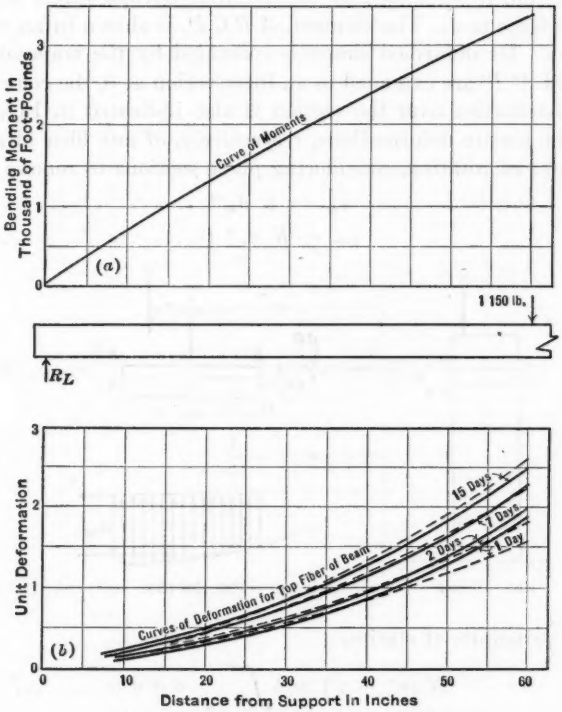

FIG. 15.—EXPERIMENTS BY E. B. SMITH.

The moment curve (Fig. 15(a)) is for combined dead and live loads. The graphs (Fig. 15(b)) have been plotted from experimental curves, indicated by Mr. Smith, by making correction for the shrinkage. Superimposed are a series of curves, in broken lines, indicating the plastic flow determined by the proposed exponential formula.

At the center of the beam, it will be noticed, the theoretical curves do not agree very closely with the experimental curves, especially for the short time intervals. It seems possible that this may be due in part to the localized effect of the concentrated load.

DERIVATION OF EXACT EQUATIONS FOR THE PLASTIC THEORY

General Requirements.—As with the analysis for a variable modulus of elasticity, so for the proposed theory of plasticity—it is not possible to superimpose the effects of individual forces separately upon a structure to obtain the effect of the combined system. In addition, the theory of plasticity presents

further difficulty. Except in special cases, the elastic deformations and plastic deformations are dependent on each other; and, to be perfectly exact, the total deformation may not be written as the sum of two explicit functions of the elastic and plastic deformation, respectively.

Derivation of Equations.—Consider a column (Fig. 16(*a*)), measuring b by d in section, to be loaded by an eccentric normal force, N. The eccentricity of the force is z'. The element, $A\,B\,C\,D$, is shown in an enlarged scale in Fig. 16(*b*). Its deformed shape is indicated by the trapezoid, $A'\,B'\,C\,D$. Lines $A\,B$ and $A'\,B'$ are extended to an intersection at 0, the center of rotation. The stress distribution over the section is also indicated in Fig. 16(*b*). The elastic and the plastic deformations, respectively, of any fiber at a distance, w, from the center of rotation, considering plane sections to remain plane, are,

$$\epsilon_w = k\ \sigma_w{}^m \qquad (33)$$

$$\rho_w = K\ \sigma_w{}^p\ t^q \qquad (34)$$

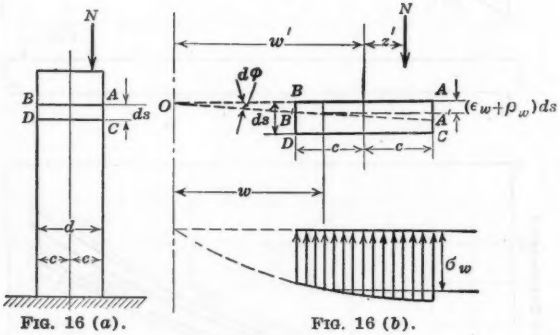

FIG. 16 (*a*). FIG. 16 (*b*).

From the principles of statics,

$$N\ (w' + z') = b \int_{w'-c}^{w'+c} \sigma\ w\ d\ w \qquad (35)$$

and,

$$N = b \int_{w'-c}^{w'+c} \sigma\ d\ w \qquad (36)$$

Thus, dividing Equation (35) by Equation (36),

$$(w' + z') = \int_{w'-c}^{w'+c} \sigma\ w\ d\ w \div \int_{w'-c}^{w'+c} \sigma\ d\ w \qquad (37)$$

From the geometry of Fig. 16(*b*) and Equations (33) and (34),

$$\left.\begin{aligned} w \frac{d\ \phi}{d\ s} &= \epsilon_w + \rho_w = k\ \sigma_w{}^m + K\ \sigma_w{}^p\ t^q \\ w &= \frac{1}{\dfrac{d\ \phi}{d\ s}} [k\ \sigma_w{}^m + K \sigma_w{}^p\ t^q] \\ d\ w &= \frac{1}{\dfrac{d\ \phi}{d\ s}}\ [m\ k\ \sigma_w{}^{m-1} + p\ K\ \sigma_w{}^{p-1}\ t^q] \end{aligned}\right\} \qquad (38)$$

Considering $\frac{d\,\phi}{d\,s}$ a parameter, values of w and $d\,w$ from Equations (38) substituted into Equations (35) and (36) result in,

$$N\,(w' + z') = \frac{1}{\left(\frac{d\,\phi}{d\,s}\right)^2}\int_{\sigma_2}^{\sigma_1} \sigma\,[k\,\sigma^m + K\,\sigma^p\,t^q]\,[m\,k\,\sigma^{m-1} + p\,K\,\sigma^{p-1}\,t^q]\,d\,\sigma$$

$$= \frac{1}{\left(\frac{d\,\phi}{d\,s}\right)^2}\left[\int_{\sigma_2}^{\sigma_1} m\,k^2\sigma^{2m}\,d\,\sigma + \int_{\sigma_2}^{\sigma_1}(m+p)\,k\,K\,t^q\,\sigma^{p+m}\,d\,\sigma + \int_{\sigma_2}^{\sigma_1} p\,K^2\,\sigma^{2p}\,t^{2q}\,d\,\sigma\right]$$

which when integrated becomes,

$$N\,(w' + z') = \frac{1}{\left(\frac{d\,\phi}{d\,s}\right)^2}\left[\frac{m\,k^2}{2\,m+1}\,(\sigma_1{}^{2\,m+1} - \sigma_2{}^{2\,m+1}) + \frac{(m+p)\,k\,K\,t^q}{(p+m+1)}\,(\sigma_1{}^{(p+m+1)} - \sigma_2{}^{(p+m+1)}) + \frac{p\,K^2\,t^{2q}}{2\,p+1}\,(\sigma_1{}^{2p+1} - \sigma_2{}^{2p+1})\right]$$

or,

$$N\,(w' + z') = \frac{F}{\left(\frac{d\,\phi}{d\,s}\right)^2} \quad \ldots\ldots(39)$$

in which,

$$F = \left[\frac{m\,k^2}{2\,m+1}\,(\sigma_1{}^{2\,m+1} - \sigma_2{}^{2\,m+1}) + \frac{(m+p)\,k\,K\,t^q}{p+m+1}\,(\sigma_1{}^{(p+m+1)} - \sigma_2{}^{(p+m+1)}) + \frac{p\,K^2\,t^{2q}}{2\,p+1}\,(\sigma_1{}^{2p+1} - \sigma_2{}^{2p+1})\right] \quad \ldots\ldots(40)$$

By a similar process it will be found that,

$$N = \frac{G}{\frac{d\,\phi}{d\,s}} \quad \ldots\ldots(41)$$

in which,

$$G = \frac{m\,k}{m+1}\,(\sigma_1{}^{m+1} - \sigma_2{}^{m+1}) + \frac{p\,K\,t^q}{p+1}\,(\sigma_1{}^{p+1} - \sigma_2{}^{p+1}) \quad \ldots\ldots(42)$$

From Equations (39) and (41) may be obtained the relation,

$$\frac{d\,\phi}{d\,s} = \frac{F}{G\,(w' + z')} \quad \ldots\ldots(43)$$

Special Case, $t = 0$.—Immediately after the load is applied, that is, the special case when $t = 0$, w' becomes equal to u' and Equation (43) reduces to,

$$\frac{d\,\phi_E}{d\,s} = k\,\frac{m+1}{2\,m+1}\,\frac{(\sigma_1{}^{2\,m+1} - \sigma_2{}^{2\,m+1})}{(\sigma_1{}^{m+1} - \sigma_2{}^{m+1})} \times \frac{1}{u' + z'} \quad \ldots\ldots(44)$$

but,

$$\sigma = \left(\frac{\varepsilon}{k}\right)^{\frac{1}{m}} \quad \ldots\ldots(45)$$

also,

$$\varepsilon_1 = (u' + c)\,\frac{d\,\phi_E}{d\,s} \text{ and } \varepsilon_2 = (u' - c)\,\frac{d\,\phi_E}{d\,s} \quad \ldots\ldots(46)$$

hence, from Equations (45) and (46),

$$\sigma_1 = \frac{\left[(u' + c)\frac{d\,\phi_E}{d\,s}\right]^{\frac{1}{m}}}{k^{\frac{1}{m}}} \text{ and } \sigma_2 = \frac{\left[(u' - c)\frac{d\,\phi_E}{d\,s}\right]^{\frac{1}{m}}}{k^{\frac{1}{m}}} \quad \ldots\ldots(47)$$

Substituting the values of σ_1 and σ_2 from Equations (47) into Equation (44) and re-arranging terms,

$$(u' + z') = \frac{m+1}{2m+1} \frac{\left[(u' + c)^{\frac{1+2m}{m}} - (u' - c)^{\frac{1+2m}{m}}\right]}{\left[(u' + c)^{\frac{1+m}{m}} - (u' - c)^{\frac{1+m}{m}}\right]} \quad \ldots\ldots(48)$$

which is identical with Equation (22) for the proposed theory of elasticity with E introduced as a variable.

Development of the Theory of Plasticity for the Arch

General Assumptions. —The total rate of angle change, $\frac{d\,\phi}{d\,s}$, given by Equation (43), is very complicated and would prove extremely unwieldly, if not impossible, to apply to a structure such as the concrete arch. The unknowns are in terms of the extreme fiber stresses which, according to the theory, will vary with time even should the load and eccentricity of the load remain constant.

To obviate this difficulty the following equation is introduced,

$$\frac{d\,\phi}{d\,s} = \frac{d\,\phi_E}{d\,s} + \frac{d\,\phi_P}{d\,s} \quad \ldots\ldots(49)$$

as a fundamental basis for further analysis. In this relation, $\frac{d\,\phi}{d\,s}$ is the total rate of rotation at a given section; $\frac{d\,\phi_E}{d\,s}$ is the rate of elastic rotation at the section; while $\frac{d\,\phi_P}{d\,s}$ is the rate of plastic rotation at the end of a particular time interval.

The theory, as here presented, is not quite exact, for it is developed on the assumption that the center of plastic rotation for each section is independent of the elastic properties of the concrete. To be perfectly exact, the center of plastic rotation at a section at the beginning of the stressed condition coincides with the center of elastic rotation, and moves toward the ultimate center of plastic rotation as the time interval of stress continues and redistribution of stresses within the section takes place. The theory is exact when the rate of change of elastic deformation with variation in stress is equal to the rate of change of plastic deformation (that is, when the parameters, m and p, are equal), because in this case no redistribution of stresses takes place, and the center of elastic rotation coincides with the center of plastic rotation.

However, since the location of the ultimate center of plastic deformation approximates that of the center of elastic deformation for the probable range of values of m and p, no large error is introduced by assuming that there is an instantaneous movement of the center of rotation from the elastic center to the ultimate plastic center.

The group of relations derived from Equation (49) will satisfy all the laws of statics and conditions of continuity; hence, they are mathematically correct. The term, $\frac{d\,\phi_E}{d\,s}$, of the second member of Equation (49) will be the same as for the proposed theory of elasticity.

Although the derivation which follows is for combined compression and flexure, Equation (49) may be expanded equally well for pure flexure.

Derivation of Simplified Equations of Plasticity.—Consider (Fig. 17(*a*)) an elementary radial segment, $d\,s$, of an arch. The center of the element is located at a point the abscissa of which is x. It is subjected to a normal thrust, N_x, acting at a distance, z', from the centroidal axis.

An enlarged diagram of the element is indicated in Fig. 17(*b*). The elastic deformation of the segment is indicated by the trapezoid, $A\,B\,B'\,A'$; the plastic deformation is $A'\,B'\,B''\,A''$. Point O is the center of elastic rotation, while Point O' is considered to be the center of plastic rotation. Other nomenclature is indicated in the diagram.

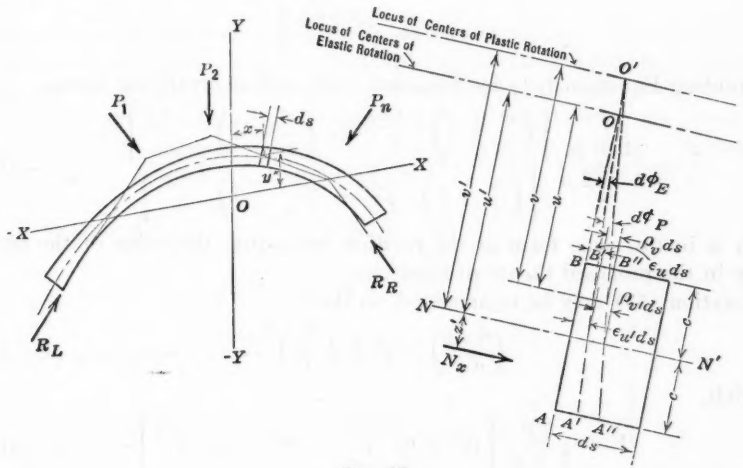

FIG. 17.

To determine $\frac{d\,\phi_P}{d\,s}$, the plastic deformation of a fiber at a distance, v, from the center of plastic rotation, O', assuming stress distribution over the section to be dependent on the plastic properties of the material, is,

$$\rho_v = K\,\sigma_v^{\,p}\,t^q \qquad (50)$$

Geometric relations show that,

$$\rho_v = v\,\frac{d\,\phi_P}{d\,s} \qquad (51)$$

Eliminating ρ_v between Equations (50) and (51), and re-arranging terms,

$$\sigma_v = \frac{1}{(t)^{\frac{q}{p}}}\left(\frac{v}{K}\,\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \qquad (52)$$

Since, for static equilibrium, the sum of the forces acting on the section in line with $n\,n'$ must be zero,

$$N_x = \int_{(v'-c)}^{(v'+c)} \sigma_v \, d\,A = \frac{b}{(t)^{\frac{q}{p}}\,K^{\frac{1}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \int_{(v'-c)}^{(v'+c)} v^{\frac{1}{p}}\,d\,v$$

from which,

$$N_x = \frac{p}{1+p} \frac{b}{K^{\frac{1}{p}}\,t^{\frac{q}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \left[(v'+c)^{\frac{1+p}{p}} - (v-c)^{\frac{1+p}{p}}\right] \ldots (53)$$

For equilibrium against static rotation,

$$N_x\,(v'+z') = \frac{b}{K^{\frac{1}{p}}\,t^{\frac{q}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \int_{(v'-c)}^{(v'+c)} v^{\frac{1+p}{p}}\,d\,v$$

which, when integrated, becomes,

$$N_x\,(v'+z') = \frac{p}{1+2\,p} \frac{b}{K^{\frac{1}{p}}\,t^{\frac{q}{p}}} \left(\frac{d\,\phi_P}{d\,s}\right)^{\frac{1}{p}} \left[(v'+c)^{\frac{1+2p}{p}} - (v'-c)^{\frac{1+2p}{p}}\right] \ldots\ldots\ldots (54)$$

Dividing Equation (54) by Equation (53), and re-arranging terms,

$$\frac{z'}{c} = \frac{1+p}{1+2\,p} \frac{\left[\left(\frac{v'}{c}+1\right)^{\frac{1+2p}{p}} - \left(\frac{v'}{c}-1\right)^{\frac{1+2p}{p}}\right]}{\left[\left(\frac{v'}{c}+1\right)^{\frac{1+p}{p}} - \left(\frac{v'}{c}-1\right)^{\frac{1+p}{p}}\right]} - \frac{v'}{c} \ldots (55)$$

which is in the same form as the relation indicating the value of the eccentricity in the proposed theory of elasticity.

Equation (53) may be re-arranged, so that,

$$\left(\frac{d\,\phi_P}{d\,s}\right) = t^q\,K \left(\frac{N_x}{V}\right)^p \ldots\ldots\ldots (56)$$

in which,

$$V = \frac{b\,p}{1+p} \left[(v'+c)^{\frac{1+p}{p}} - (v'-c)^{\frac{1+p}{p}}\right] \ldots\ldots\ldots (57)$$

Equations (56) and (57) are of the same form as Equations (24) and (25) in the proposed theory of elasticity. Equation (56) may also be written in the form,

$$\left(\frac{d\,\phi_P}{d\,s}\right) = t^q\,K\,\frac{M^p}{(z'\,V)^p} \ldots\ldots\ldots (58)$$

The rib-shortening of the elemental length, $d\,s$, due to plastic flow, is,

$$d\,\xi_p = \rho_{v'}\,d\,s$$

Geometrically,

$$\rho_{v'} = v'\,\frac{d\,\phi_P}{d\,s}$$

hence,

$$\frac{d\,\xi_P}{d\,s} = v'\,\frac{d\,\phi_P}{d\,s}$$

or,

$$\frac{d\,\xi_P}{d\,s} = v'\,t^q\,K\left(\frac{N_x}{V}\right)^p \qquad (59)$$

The total rate of rotation at a section of the arch is now given by the expression,

$$\frac{d\,\phi}{d\,s} = \frac{d\,\phi_E}{d\,s} + \frac{d\,\phi_P}{d\,s}$$

or,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{N_x}{U}\right)^m + t^q\,K\left(\frac{N_x}{V}\right)^p \qquad (60)$$

Also, the total rate of rib-shortening, is,

$$\frac{d\,\xi}{d\,s} = \frac{d\,\xi_E}{d\,s} + \frac{d\,\xi_P}{d\,s}$$

or,

$$\frac{d\,\xi}{d\,s} = u'\,k\left(\frac{N_x}{U}\right)^m + v'\,t^q\,K\left(\frac{N_x}{V}\right)^p \qquad (61)$$

Consider, now, a symmetrical arch, and neglect shear deformations. The dead load on the arch will be symmetrical; hence, rib-shortening can cause no tendency for relative vertical displacement of the abutments. Substituting Equations (60) and (61) into Equations (5), which indicate the geometrical relations of the change in shape of the arch, the following equations are obtained.

$$\left.\begin{aligned} 0 &= k\int\left(\frac{N}{U}\right)^m \frac{y\,d\,x}{\cos\alpha} + t^q\,K\int\left(\frac{N}{V}\right)^p \frac{y\,d\,x}{\cos\alpha} \\ &\quad + k\int u'\left(\frac{N}{U}\right)^m d\,x + t^q\,K\int v'\left(\frac{N}{V}\right)^p d\,x \\ 0 &= k\int\left(\frac{N}{U}\right)^m \frac{x\,d\,x}{\cos\alpha} + t^q\,K\int\left(\frac{N}{V}\right)^p \frac{x\,d\,x}{\cos\alpha} \\ 0 &= k\int\left(\frac{N}{U}\right)^m \frac{d\,x}{\cos\alpha} + t^q\,K\int\left(\frac{N}{V}\right)^p \frac{d\,x}{\cos\alpha} \end{aligned}\right\} \qquad (62)$$

While it is entirely possible to combine these equations with the static relations, the result would be exceedingly complicated. Since Maxwell's law of reciprocal deflections has no meaning in these equations, no simplification can be introduced by special arrangement of the co-ordinate system, or the like.

Equations (62) are derived on the basis of a statically determinate structure, that is, the location of the line of thrust in the arch must be known before the angle changes can be computed. For example,

$$t^q\,K\int\left(\frac{N}{V}\right)^p \frac{y\,d\,x}{\cos\alpha}$$

is the relative movement of the ends of the imaginary cantilevers terminating at the center of co-ordinates, along the x-axis.

Equations (62) may be solved with the aid of Figs. 9 and 10 in a manner similar to that explained for the proposed theory of elasticity.

Effect of Plastic Flow in an Arch When $m = p$.—If m and p are equal to the same number, n, that is, $n = m = p$, it will be found there are common values, w and w', such that $V = U = V$ and $w' = u' = v'$.

Equations (62) may then be written in the form,

$$\left.\begin{aligned} 0 &= (k + t^q K)\left\{\int_R^L \left(\frac{N}{W}\right)^n \frac{y\,d\,x}{\cos\alpha} + \int_R^L \left(\frac{N}{W}\right)^n w'\,d\,x\right\} \\ 0 &= (k + t^q K)\int_R^L \left(\frac{N}{W}\right)^n \frac{x\,d\,x}{\cos\alpha} \\ 0 &= (k + t^q K)\int_R^L \left(\frac{N}{W}\right)^n \frac{d\,x}{\cos\alpha} \end{aligned}\right\} \quad \ldots\ldots(63)$$

It will be noted from Equations (63) that, in case the exponents, m and p, are equal, there can be no relative movement of the line of thrust due to plastic flow. Thus, although an arch undergoes a comparatively large amount of rib-shortening within a time interval, because of flow of the concrete, this is no reason for concluding that there has been a large redistribution of moments or stresses.

An actual structure, of course, presents greater complications than those indicated, for shrinkage, temperature changes, and live loads of varying intensity act in conjunction with the constant dead load pressures.

The Theory of Plasticity for Pure Flexure

General Equation.—The rate of angle change at a section of a member subjected to both direct and flexural stresses is dependent on the intensity of the direct thrust as well as on the bending moment. Hence, Equation (49) takes a special form when developed for flexure alone.

A derivation of the flexure formulas for plasticity is similar to the development presented in the proposed theory of elasticity for flexure. It will be found that,

$$\frac{d\,\phi_P}{d\,s} = K\left(\frac{M}{S}\right)^p t^q \quad \ldots\ldots(64)$$

in which,

$$S = \frac{2\,p}{1 + 2\,p}\,b\,c^{\frac{1+2p}{p}} \quad \ldots\ldots(65)$$

Other terms in Equation (64) have been defined in the development of the theory of plasticity for arches.

Substituting the values of $\frac{d\,\phi_E}{d\,s}$ and $\frac{d\,\phi_P}{d\,s}$, found, respectively, in Equations (13) and (64), into Equation (49), the relation for the total rate of angle change becomes,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q \quad \ldots\ldots(66)$$

As in the case of the theory of plasticity for arches, this relation is physically exact only when $n = m = p$. In that case, $Z = Q = S$, and Equation (66) may be rewritten as,

$$\frac{d\,\phi}{d\,s} = (k + t^q K)\left(\frac{M}{Z}\right)^n \quad \ldots\ldots(67)$$

IV.—ANALYSIS OF STRUCTURES IN PURE FLEXURE

Concrete structures designed for flexure can be economical only when combined with steel, for concrete has a very low tensile strength. However, if fiber stresses were kept sufficiently low, it would be entirely possible to design concrete beams to support loads by pure flexure.

What follows covers studies of beams composed of a material having elastic and plastic properties similar to that indicated by the experimental data on concrete, and shows their application to structures in pure flexure and the effects to be expected because of such phenomena as plastic flow. In addition, computations have been made indicating what effect a variable relation of stress to strain has on the distribution of stress and moments in a loaded structure.

No particular attempt has been made to choose exact constants to define the properties of the material in the exponential equations, but values have been chosen which will probably give the extreme conditions found in practice. Before very close values can be assigned to the parameters in the exponential equations, more precise experiments must be performed. This is especially true for the proposed plastic equations.

Deflection of Simple Supported Beam with Concentrated Load at Center.—Fig. 18 indicates a simple supported beam, of length, l, subjected to a concentrated load, P, at mid-span. The beam will be assumed to be 10 in. wide and 40 in. deep. Consider that both elastic and plastic deformations take place under the influence of the load, and that the parameters in the exponential equations have the following values: $m = p = n = 1.15$; $k = 183 \times 10^{-9}$; $K = 17 \times 10^{-9}$; and $q = 0.40$.

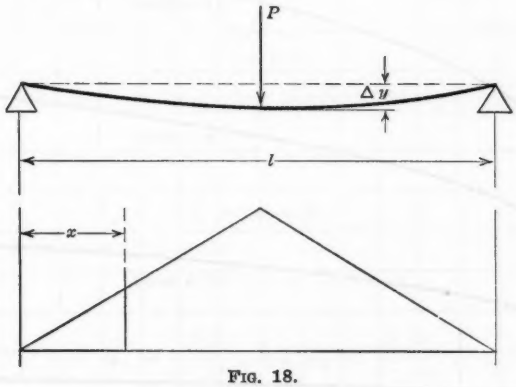

FIG. 18.

Since the beam is symmetrical, the tangent to the beam under the load will remain parallel to a line through the supports. Hence, the center deflection may be expressed by,

$$\Delta y = \int_0^{\frac{1}{2}l} \frac{d\phi}{ds} x \, dx$$

but,

$$\frac{d\,\phi}{d\,s} = (k + t^q\,K)\left(\frac{M}{Z}\right)^n$$

The bending moment at any point along the beam is,

$$M = \frac{1}{2}\,P\,x$$

hence,

$$\Delta\,y = (k + t^q\,K)\left(\frac{\frac{1}{2}\,P}{Z}\right)^n \int_0^{\frac{1}{2}\,l} x^{(1+n)}\,d\,x$$

Integrating and substituting numerical values, where given,

$$\Delta\,y = q^{1.15}\,\frac{(183 + 17\,t^{0.4})\,10^{-13}}{1\,135}\,l^{3.15}$$

In order that numerical results may be obtained, the length, l, of the beam will be considered to be 250 in. The last equation may then be rewritten as,

$$\Delta\,y = P^{1.15}\,(57.2 + 5.32\,t^{0.4})\,10^{-8}$$

The deflections at the ends of various time intervals and for different magnitudes of the concentrated load, are plotted in Fig. 19. The ordinates corresponding to the respective load concentrations on the line of zero abscissas measure the deflection at the center of the beam due to elastic deformations produced at the time the load is applied.

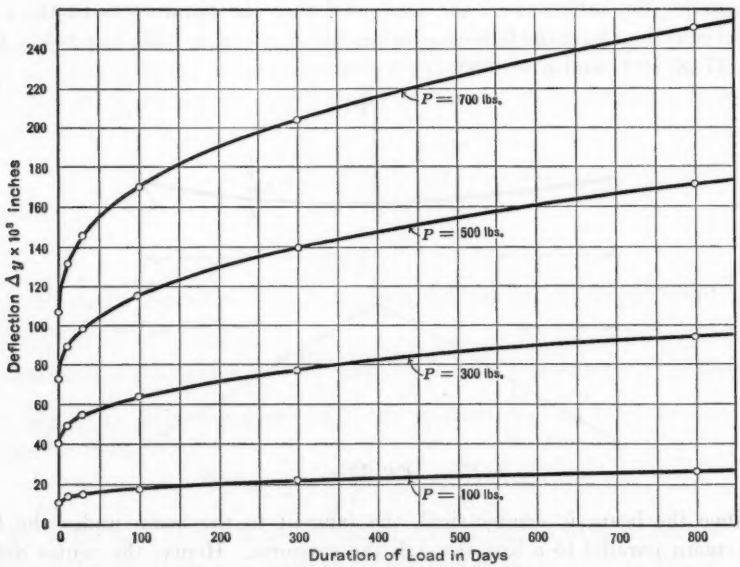

FIG. 19.—PLASTIC DEFLECTION OF BEAM.

Beam Fixed at One End and Supported Unrestrained at the Other End.—Consider the beam, $A\,B$ (Fig. 20 (*a*)). The moment diagram (Fig. 20 (*b*))

is made up of the moment diagram for a simple beam loaded with the force, P, and the diagram due to the fixing of the beam at A.

The mathematics involved in finding the moments is considerably simplified by replacing the moment and shear at A (Fig. 20(c)) by a force, R_L (for convenience in visualizing assumed to act upon the end of a rigid lever arm fastened to the beam at A), applied at the center of the rotation, such that,

$$M_A = R_L\,(a - x') \text{ and } V^A = R_L$$

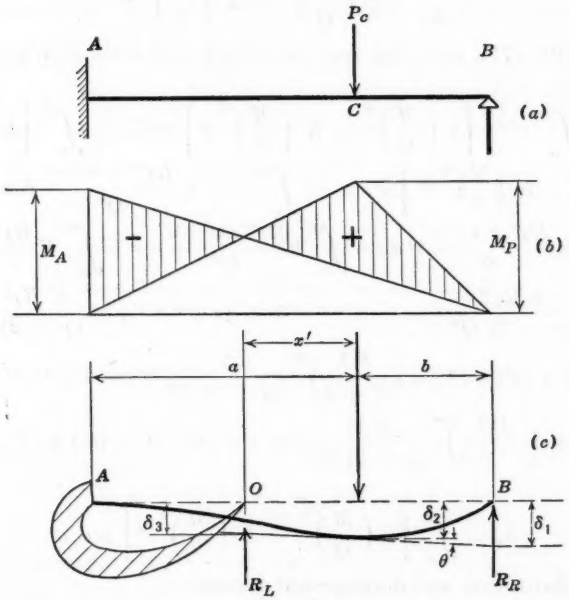

FIG. 20.

If θ is the rotation of the tangent to the beam at the concentrated load, P, by geometry,

$$\delta_2 = \delta_1 - \theta\, b$$

and,

$$\delta_2 = \delta_3 + \theta\, x'$$

Subtracting and re-arranging terms,

$$\delta_1 - \delta_3 - (x' + b)\,\theta = 0 \ldots\ldots\ldots\ldots\ldots\ldots (68)$$

By statics,

$$R_L = \frac{P\,b}{x' + b}; \text{ and } R_R = \frac{P\,x'}{x' + b} \ldots\ldots\ldots\ldots\ldots\ldots (69)$$

Since δ_3 is the deflection of O away from the tangent at O,

$$\delta_3 = \int_0^{(a-x')} \frac{d\,\phi}{d\,s}\, x\, d\,x + \int_0^{x'} \frac{d\,\phi}{d\,s}\, x\, d\,x \ldots\ldots\ldots\ldots\ldots (70)$$

Similarly, δ_1 is the deflection of B away from the tangent at C, hence,

$$\delta_1 = \int_0^{b} \frac{d\,\phi}{d\,s}\, x\, d\,x \ldots\ldots\ldots\ldots\ldots\ldots (71)$$

The slope of the tangent at C is,

$$\theta = \int_{x'}^{(a-x')} \frac{d\,\phi}{d\,s}\, d\,x \ldots\ldots\ldots\ldots\ldots\ldots\ldots (72)$$

In Equations (70) and (72) the center of co-ordinates is at O; in Equation (71), B is the center of co-ordinates.

Since,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q$$

Equations (70), (71), and (72) may be written and developed, respectively, as follows:

$$\delta_3 = \int_0^{(a-x')} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] x\, d\,x + \int_0^{x'} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] x\, d\,x = \int_0^{(a-x')} k\,\frac{R_L{}^m\, x^{(m+1)}}{Q^m}\, d\,x$$

$$+ \int_0^{(a-x')} K\,\frac{R_L{}^p\, x^{(p+1)}}{S^p}\, t^q\, d\,x + \int_0^{x'} k\,\frac{R_L{}^m\, x^{(m+1)}}{Q^m}\, d\,x + \int_0^{x'} K\,\frac{R_L{}^p\, x^{(p+1)}}{S^p}\, t^q\, d\,x$$

$$= \frac{k\,R_L{}^m}{(m+2)\,Q^m}\left[(a-x')^{(m+2)} + (x')^{(m+2)}\right] + \frac{K\,R_L{}^p\, t^q}{(p+2)\,S^p}$$

$$\left[(a-x')^{(p+2)} + (x')^{(p+2)}\right] = \left(\frac{P\,b}{x'+b}\right)^m \frac{k}{(m+2)\,Q^m}\left[(a-x')^{(m+2)} + (x')^{(m+2)}\right]$$

$$+ \left(\frac{P\,b}{x'+b}\right)^p \frac{K\,t^q}{(p+2)\,S^p}\left[(a-x')^{(p+2)} + (x')^{(p+2)}\right] \ldots\ldots\ldots (73)$$

also,

$$\delta_1 = \int_0^b \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] x\, d\,x$$

by similar substitution and development becomes,

$$\delta_1 = k\left(\frac{P\,b}{x'+b}\right)^m \frac{(x')^m\, b^2}{(m+2)\,Q^m} + K\left(\frac{P\,b}{x'+b}\right)^p \frac{(x')^p\, b^2\, t^q}{(p+2)\,S^p} \ldots\ldots (74)$$

Similarly,

$$\theta = \int_{x'}^{(a-x')} \left[k\left(\frac{M}{Q}\right)^m + K\left(\frac{M}{S}\right)^p t^q\right] d\,x$$

which becomes,

$$\theta = \left(\frac{P\,b}{x'+b}\right)^m \frac{k}{(m+1)\,Q^m}\left[(a-x')^{(m+1)} - (x')^{(m+1)}\right]$$

$$+ \left(\frac{P\,b}{x'+b}\right)^p \frac{K\,t^q}{(p+1)\,S^p}\left[(a-x')^{(p+1)} - (x')^{(p+1)}\right] \ldots\ldots (75)$$

By substituting the values from Equations (73), (74), and (75) into Equations (68), and collecting terms,

$$k\left[\frac{P\,b}{(x'+b)\,Q}\right]^m \left\{\frac{(x')^m\, b^2}{(m+2)} - \frac{\left[(a-x')^{m+2} + (x')^{m+2}\right]}{(m+2)}\right.$$

$$\left. - \frac{x'+b}{m+1}\left[(a-x')^{(m+1)} - (x')^{(m+1)}\right]\right\} + K\,t^q\left[\frac{P\,b}{(x'+b)\,S}\right]^p \left\{\frac{(x')^p\, b^2}{(p+2)}\right.$$

$$\left. - \frac{\left[(a-x')^{p+2} + (x')^{p+2}\right]}{(p+2)} - \frac{x+b}{p+2}\left[(a-x')^{p+1} - (x')^{p+1}\right]\right\} = 0 \ldots (76)$$

At the beginning of a loaded condition there is no plastic flow; $t = 0$, and hence,

$$\frac{(x')^m b^2}{(m+2)} - \frac{[(a-x')^{m+2} + (x')^{m+2}]}{(m+2)} - \frac{x'+b}{(m+1)}[(a-x')^{m+1} - (x')^{m+1}] = 0 \quad \ldots\ldots\ldots\ldots (77)$$

Special Case, in Which $m = 1$ and $a = b = \frac{1}{2} l$.—This is the condition of a beam of homogeneous material following Hooke's law, and loaded at mid-span. The beam is indicated in Fig. 21(*a*).

By the ordinary theory of elasticity, the statical moment of the area under the moment curve about B (Fig. 21(*b*)) is equal to the deflection of B away from the tangent at A, hence, by simple statics, it will be found that,

$$x' = \frac{5}{22} l$$

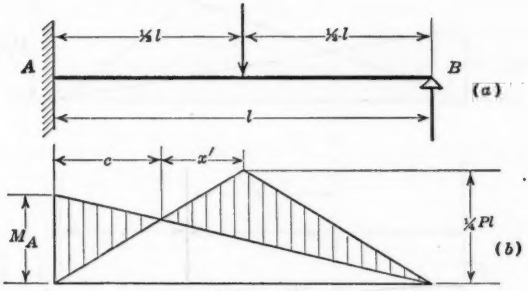

FIG. 21.

A test of Equation (77) is obtained by substituting into the equation the values of c and x' as obtained from the ordinary theory of elasticity. The first member of the equation reduces to zero, thereby verifying the derived equation.

Special Case, in Which $m = 1.3$, and $a = b = \frac{1}{2} l$.—This is an exaggerated case for concrete in which Hooke's law does not apply. The location of the point of contraflexure will be assumed to be the same as that found by the theory of elasticity (stress is proportional to strain). A substitution of the assumed values, $m = 1.3$, into Equation (77) again reduces the expression practically to an identity.

Thus, Equation (77) is satisfied for values of m other than 1, without appreciably moving the point of contraflexure. Hence, one might conclude that the moments in the beam are not affected by static loads, although the modulus of elasticity of the material varies with the stress.

Further, an analysis of the first term of Equation (76) indicates that the flow of the material with the lapse of time does not change the moment distribution in the beam. It will be observed that both members of the first term contain factors similar to the first term of Equation (77). Hence, it may be concluded that Equation (76) will be satisfied by a value of x' which would be obtained by the ordinary theory of elasticity in which Hooke's law is as-

sumed to be correct. This conclusion, of course, is on the basis of a homogeneous material.

Although the distribution of moments in the beam remains the same throughout any time interval, the distribution of stresses and the configuration of the beam will constantly change as the time interval increases.

Beam Fixed at One End, Supported Unrestrained at the Other End, and Free Support Allowed to Settle a Distance, Δy.—The support at B, in Fig. 22(a), is assumed to settle an arbitrary amount, Δy, when the load, P, is applied.

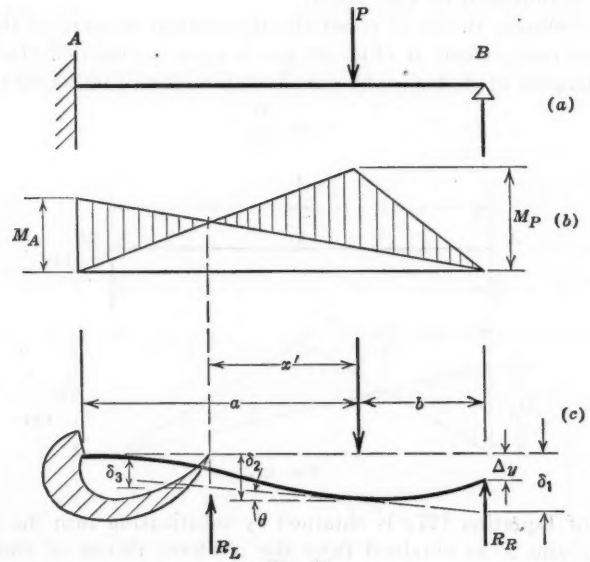

FIG. 22.

Equation (74) then takes the form,

$$\sigma_1 = k\left(\frac{P\,b}{x'+b}\right)^m \frac{(x')^m\,b^2}{(m+2)\,Q^m} + K\left(\frac{P\,b}{x'+b}\right)^p \frac{(x')^p\,b^2\,t^q}{(p+2)\,S^p} + \Delta y$$

This results in a modification of Equation (76), which then may be written as,

$$t^q = \frac{-k\left[\frac{P\,b}{(x'+b)\,Q}\right]^m \left\{\frac{(x')^m\,b^2}{(m+2)} - \frac{[(a-x')^{m+2}+(x')^{m+2}]}{(m+2)} - \frac{x'+b}{(m+1)}[(a-x')^{m+1}-(x')^{m+1}]\right\} - \Delta y}{K\left[\frac{P\,b}{(x'+b)\,S}\right]^p \left\{\frac{(x')^p\,b^2}{(p+2)} - \frac{[(a-x')^{p+2}+(x')^{p+2}]}{(p+2)} - \frac{(x'+b)}{(p+1)}[(a-x')^{p+1}-(x')^{p+1}]\right\}} \quad \ldots (78)$$

A scrutiny of Equation (78) immediately discloses that the distribution of moments in the beam changes with the time. A state of equilibrium is approached as the time interval, t, becomes very large.

When x' in Equation (78) becomes equal to the value of x' which satisfies Equation (76), the denominator of the second member of Equation (78) becomes zero; hence, t becomes infinite. This indicates that, with a lapse of time, the moment distribution in the beam approaches that of the beam without settlement of the support.

Immediately after the load, P, is applied with resulting settlement of support, $t = 0$; hence, Equation (78) takes the form,

$$k\left[\frac{P\,b}{(x'+b)\,Q}\right]^m \left\{\frac{(x')^m\,b^2}{(m+2)} - \frac{[(a-x')^{(m+2)} + (x')^{(m+2)}]}{(m+2)} - \frac{x'+b}{(m+1)}[(a-x')^{(m+1)} - (x')^{(m+1)}]\right\} + \Delta y = 0 \ldots\ldots (79)$$

The distribution of moments then varies as a function of the time interval. The limiting values of x' will be that just after loading and settling of the support and that for the conditions of no settlement of the support. The equations have been derived assuming a statically determinate condition, which, of course, is not directly applicable to indeterminate structures. That is, the equations are so set up that one may find the change in shape of the structure at the end of any time interval provided the moment distribution is known.

Special Case in Which Hooke's Law Is True.—Assuming that Hooke's law applies, that is, $m = 1$, Equation (79) becomes,

$$\Delta y = \frac{P\,b}{(x'+b)\,E\,I}\left\{\frac{x'\,b^2}{3} - \frac{1}{3}[(a-x')^3 + (x')^3] - \frac{1}{2}(x'+b)[(a-x')^2 - (x')^2]\right\} \ldots\ldots\ldots\ldots (80)$$

Numerical Case.—As a simple case for the study of the effects of plastic flow, consider the beam in Fig. 23. The following values will be considered to satisfy the properties of the concrete in the exponential equations: $m = 1.15$; $p = 1.30$; $q = 0.40$; $K = 36 \times 10^{-9}$; and $k = 182 \times 10^{-9}$.

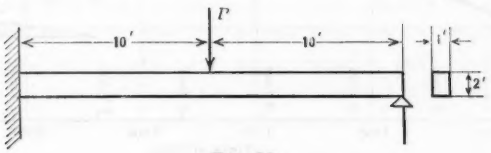

FIG. 23.

Assume that the right support settles at the instant of applying the load such that the right reaction is zero and the beam is just in contact with the support. In Equation (78) x' will be equal to zero at the beginning of the loaded condition. Hence, the settlement of the support will be,

$$\Delta y = k\left(\frac{P}{Q}\right)^m\left[\frac{a^{(m+2)}}{m+2} + \frac{b\,a^{(m+1)}}{m+1}\right]$$

or, substituting the numerical values,

$$\Delta y = P^{1.15} \times 12 \times 10^{-6} \text{ in.}$$

Assume, now, that plastic flow takes place. Deformation will take place in the part of the beam that is subject to stress, and this will result in load being transferred to the right reaction which, it is assumed, is fixed in location, that is, it does not settle further.

Considering a given location of the point of contraflexure in the beam, a study will be made to determine the time interval required for the beam to assume a conformation which will satisfy its end conditions. Thus, when a load, P, is applied at the center of the beam and the deflection at the left support is $\Delta y = P^{1.15} \times 12 \times 10^{-6}$, the point of contraflexure might be considered to be directly under the load, since the moment is zero in the right half of the beam.

If the point of inflection is considered to be a distance, x', to the left of the load, the time required for the slope of the beam at the left support to reach its normal condition may be computed by Equation (78), and would be,

$$t = \left\{ \frac{-kS^p \left\{ \left(\frac{b}{x'+b}\right)^m \left[\frac{(x')^m b^2}{m+2} - \frac{[(a-x')^{m+2} + (x')^{m+2}]}{m+2} - \frac{x'+b}{m+1}[(a-x')^{m+1} - (x')^{m+1}]\right] + \left[\frac{a^{(m+2)}}{m+2} + \frac{b\,a^{(m+1)}}{m+1}\right]\right\}}{P^{(m-p)} K Q^m \left(\frac{b}{x'+b}\right)^p \left\{ \frac{(x')^p b^2}{p+2} - \frac{[(a-x')^{p+2} + (x')^{p+2}]}{p+2} - \frac{x'+b}{p+1}[(a-x')^{p+1} - (x')^{p+1}]\right\}} \right\}^{\frac{1}{q}} \quad (81)$$

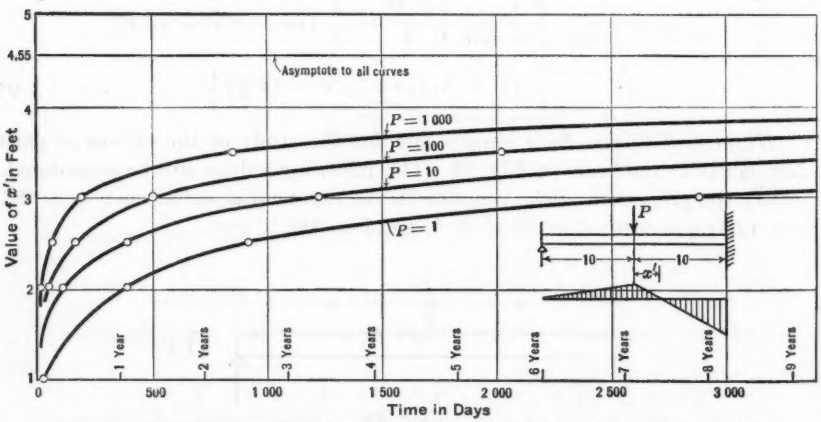

FIG. 24.—MOVEMENT OF POINT OF CENTER OF FLEXURE FOR CONCENTRATED LOAD.

Equation (81) has been solved by substituting various numerical values for the distance, x', and for the load, P. All computations for this analysis were made on an 8-in. slide-rule. A graphical representation of the results obtained by studying the movement of the point of contraflexure is shown on Fig. 24. Curves are plotted for various values of the concentrated loads, P. A load of 1 000 lb. produces an extreme fiber stress about equal to the maximum tensile strength obtained in concrete.

When the point of contraflexure approaches a location 4.55 ft. from the center of the beam, the numerator in the second member of Equation (81)

becomes zero and the time interval becomes infinite. This "critical point" is at the point of contraflexure which would be found for the condition of pure elastic deformation and no settlement of the support. Fig. 24 shows a line through the critical point as the asymptote to all curves indicating the movement of the point of contraflexure.

The variation of moment distribution as x' changes may readily be computed from simple static and geometric relations in the moment diagram. Curves have been plotted showing the moment redistribution in the beam in which the support has settled and plastic flow has occurred. Fig. 25 shows the curves considering, respectively, two different amounts of settlement of the support, A, and the load to be 1 000 lb.

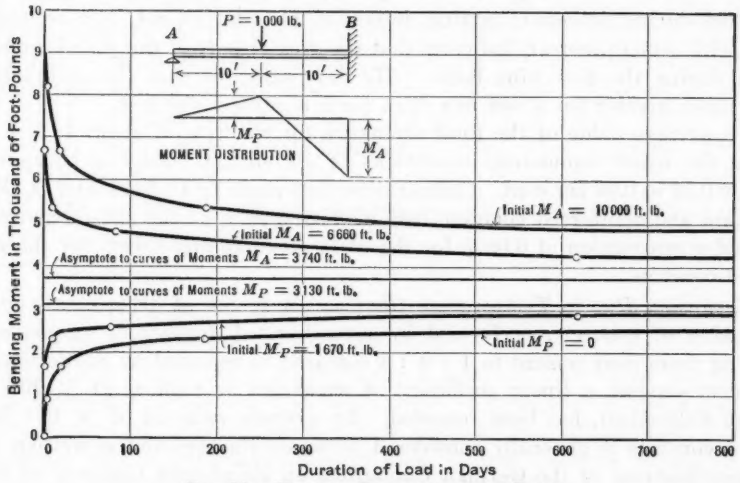

FIG. 25.—MOMENT DISTRIBUTION IN BEAM.

The graphs for the rate of change of the moment curve and point of contraflexure, of course, are subject to the criticism that the gradual changes in the moment curve have not been considered. Such progressive change can be accounted for by successive approximations and would result in the beam approaching a state of equilibrium more rapidly than indicated by the diagrams. However, this refinement is not warranted with the present status of knowledge of the plastic properties of concrete.

V.—ANALYSIS OF ARCHES

Choice of Parameters in the Proposed Theories.—Since no experimental studies of concrete have been made with the particular intention of fitting the results into theories such as the writer has proposed, it would be very strange, to say the least, to find many data which can be used directly to define numerically the parameters in the proposed equations. In general, the duration of the time-yield tests has been too short to give precise results. Usually, especially in foreign experiments, plastic flow has not been distinguished from shrinkage. In their studies of arches, the French simply combine the plastic flow and shrinkage into one effect, calling it *retrait*.

Nevertheless, since a study shows that plastic deformations on the one hand, and shrinkage and temperature deformations on the other, produce very different effects on the stresses in a structure, it becomes quite important that these properties be differentiated.

Shrinkage of Concrete.—Shrinkage is distinguished from plastic flow, in that it is a change in dimensions independent of external forces acting on the specimen.

The shrinkage of concrete varies within such wide limits that it is practically impossible, at present, to assign a definite value or law to its coefficient. Among the many factors* influencing this property are: The composition of the cement, quality of aggregate, proportioning of mix, wetness of mix, temperature during process of setting, humidity, etc. Otto Graf, in a report† on tests with cement mortar, indicates that the greater part of the shrinkage takes place during the first nine hours. He also indicates that the shrinkage is very much greater for a wet mix than for a relatively dry mix.

An average value of the total shrinkage for concrete of about 1:2:4 mix under the usual commercial conditions of air curing, seems to be approximately 0.02 to 0.03 per cent. Comparative tests made by C. Bach and O. Graf‡ on plain and reinforced concrete bars of 1:3:2 mix, at the end of five years showed a contraction of 0.05% for the plain concrete and 0.02% for the reinforced concrete.

Expansion Due to Temperature Change.—A review of experiments on the expansion of concrete, performed by several noted authorities on materials ranging from neat cement to 1 : 2 : 4 concrete, is reported by Shitkewitsch.§ For neat cement, a linear coefficient of expansion as high as 81×10^{-7} per degree Fahrenheit, has been recorded. An average value of 55×10^{-7} for a 1:2:4 concrete is generally considered to be satisfactory and is written into the specifications of the German Committee on Reinforced Concrete of 1925, and also into the specifications of the Belgian Standards Association of 1923. This value is corroborated in experiments by W. D. Pence, M. Am. Soc. C. E.‖

Within the variation of the ordinary atmospheric temperatures, the coefficient of expansion is practically constant. Indeed, for the study of stresses in concrete arches, the coefficient of conductivity through the concrete is of tremendously greater importance than the variation found in the coefficient of expansion.

Choice of Parameters for Exponential Formulas Defining Plastic Deformation in Arches.—Although the exponents, p and q, may vary considerably with concretes of various ages and qualities, the writer believes that they might be predetermined with a fair degree of precision by a systematic series of experiments. In a concrete which is not more than 2 weeks old, it appears that q might be as low as 0.15, while p might reach 2.0, or more. Concrete of a 1:2:4 mix subjected to a constant load beginning at about the age of

* "Einflüsse auf Beton", A. Kleinlogel, p. 314, Berlin, 1925.

† *Beton und Eisen*, 1921-22.

‡ "Armierter Beton", Bach and Graf, 1909.

§ "Monolität d. Betonbauten", N. A. Shitkewitsch, *Forscherarbeiten* a. d. Gebiete d. Eisenbetons, Heft 7, 1906.

‖ *Journal*, West. Soc. of Engrs., Vol. VI, p. 549, 1901.

4 months, would probably deform according to the exponential equation with q and p, respectively, equal to about 0.4 and 1.25.

From observations made on a large number of arches, E. Freyssinet, a French engineer who has developed mechanical methods of caring for rib-shortening effects in arches, gives values* for the shortening of a rib over a long period of time. The unit shortenings are listed for various cement contents as follows:

350 kg. per cu. m. (1.56 bbl. per cu. yd.)..... 4×10^{-4} to 6×10^{-4}
400 " " " " (1.78 " " " ")..... 5×10^{-4} to 7×10^{-4}
450 " " " " (2.00 " " " ")..... 6×10^{-4} to 8×10^{-4}

A curve representing the shortening over a long period of time would fluctuate above and below a mean which would slowly approach zero. The fluctuation, of course, is dependent on the temperature and humidity variations.

Assuming that, in the arches observed by Freyssinet, the shrinkage of the concrete due to setting was about 0.0003, the plastic shortening of the arch ribs made of the richer concrete would be about 0.0007 — 0.0003 = 0.0004. Hence, for an analysis of an arch in which the centering is moved within about four months after pouring the rib, the exponential equation for plastic flow would probably approximately take the form,

$$\epsilon = 17 \times 10^{-9}\, \sigma^{1.25}\, t^{0.4} \qquad (82)$$

in which, σ is the stress, in pounds per square inch, and t is the time, in days, after striking the centering. This would result in a plastic flow rib-shortening of about 0.00035 in 2 years, if the average dead load stress in the arch is 350 lb. per sq. in. According to Equation (82), it would require almost 10 years more to double this rib-shortening. It is probable that the exponential equation would take a somewhat different form as the concrete becomes older; however, no definite determination can be made from the experimental data now available.

The numerical parameters given in Equation (82) will not be strictly adhered to, in the studies which follow, but the constants chosen will be based on considerations similar to those just presented.

Description of Arches Analyzed

Description of Arch No. 1.—Arch No. 1 is a symmetrical solid arch barrel having a rise ratio of 1:4. The center line of the arch is a parabola with a span of 140 ft., and a rise of 35 ft. from springing to crown. The thickness of the rib is 2 ft. at the crown and varies as the square of the secant of the angle between the tangent to the arch center line and the horizontal. Various assumptions have been made with regard to the properties of the material composing the arch. These variations will be explained when they are introduced into the discussion and the varied analyses are carried to completion. As far as possible, all variations are based on rational grounds which are borne out by experiment.

* "Le Pont de Villeneuve-sur-Lot", M. Freyssinet, *Le Genie Civil*, v. 79, p. 126, August, 1921.

Description of Arch No. 2.—Arch No. 2 is practically the same as Arch No. 1, except that the rise ratio is 1:8 instead of 1:4.

General Analysis of the Arch.—The arch rib (Fig. 26) is divided into twenty equal horizontal lengths, $\Delta x = 7$ ft. A point half way between the ends of each voussoir was located and numbered, the numbering being from left to right. Thus, Arch-Point No. 1 is a point on the center line of the arch, 3½ ft. horizontally to the right of the left springing; Point No. 2 is 10½ ft. horizontally to the right, etc. All distances are measured in feet.

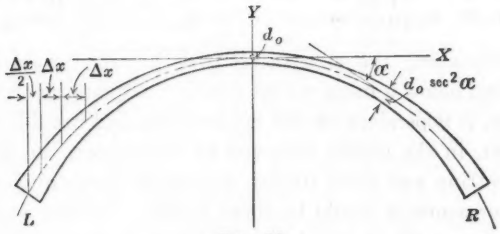

FIG. 26.

Deformations of the rib were found by computing the rate of angle change, $\frac{d\phi}{ds}$, and rate of rib-shortening, $\frac{d\xi}{ds}$, respectively, at each arch point, and assuming the total deformation within a voussoir to be concentrated at the corresponding arch point. Whenever possible, constant factors have been omitted from the computations, and introduced only where necessary to obtain relative results. In cases where logarithms were needed for numerical analysis, a constant factor was sometimes introduced to simplify the computation.

The center line of the parabolic arch with respect to the crown as the center of a rectangular co-ordinate system, is given by the equation,

$$y = \frac{x^2}{2k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (83)$$

in which, k is a parameter equal to the distance from the focus to the directrix of the parabola, or twice the distance from the focus to the crown or apex of the parabola. For the arch having a rise ratio of 1:4 and a 140-ft. span, $k = 70$ ft. The co-ordinates of the arch points were computed by Equation (83).

The slope of the tangent to the center line of the rib at any point is the first derivative of y with respect to x, hence,

$$\tan \alpha = \frac{dy}{dx} = -\frac{x}{k}$$

The cosine of the angle, α, was read from a table of trigonometric functions. The thickness of the rib at any point is,

$$d = 2c = \frac{2c_0}{\cos^2 \alpha}$$

in which, c_o is equal to one-half the crown thickness. Hence, since the thickness at the crown of the arch is 2, the vertical location of the neutral point, according to the ordinary theory of elasticity is,

$$y_c' = \frac{\sum_R^L \frac{y_c \, \Delta x}{E \, I \cos \alpha}}{\sum_R^L \frac{\Delta x}{E \, I \cos \alpha}} = \frac{\sum_R^L y_c \cos^5 \alpha \, \Delta x}{\sum_R^L \cos^5 \alpha \, \Delta x}$$

in which, y_c is the distance from the crown of the arch to an arch point. The distance from the neutral point to an arch point is then,

$$y = y_c - y_c'$$

Table 1 contains the computations for the location of the neutral point and the co-ordinates of the arch points. Then, $y_c' = \frac{41.21}{5.893} = 7.00$.

Studies of the Effects of a Variable Modulus of Elasticity on the Action of Arches

Elastic Arch Uniformly Loaded and Rib-Shortening for All Causes.—Consider a parabolic arch of the form, Arch No. 1. Fig. 27 shows the center line or axis of the arch, $L\,R$, as a solid line.

Suppose the arch to be uniformly loaded. Considering only flexural deformations in the arch, neglecting rib-shortening entirely, the line of pressure will be a parabola coinciding with the axis of the rib. Taking into account all causes except plastic flow, that is, elastic rib-shortening, temperature change, and shrinkage, there results a rib-shortening thrust, H_{RS}, which must act through the neutral point. Hence, at any point in the arch, the bending moment will be $M = H_{RS}\, y$.

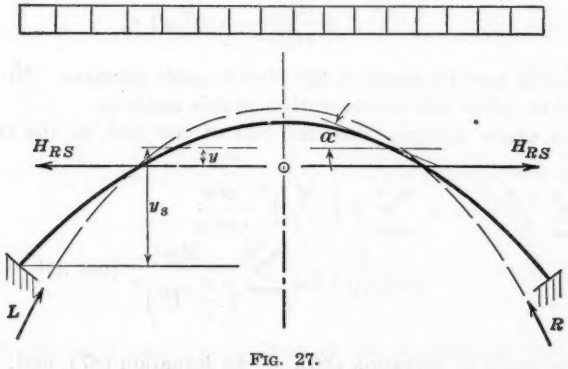

Fig. 27.

If T_{DL} is the thrust due to the uniformly distributed load, the total thrust at any section of the rib is, $T = T_{DL} - H_{RS} \cos \alpha$.

The eccentricity of the thrust line will be;

$$z' = \frac{M}{T} = \frac{H_{RS}\, y}{T_{DL} - H_{RS} \cos \alpha} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (84)$$

or,

$$z' = \frac{y}{\frac{T_{DL}}{H_{RS}} - \cos \alpha}$$

Since the arch is parabolic and uniformly loaded, the dead load thrust may be written in terms of the horizontal thrust,

$$T_{DL} = \frac{H_{DL}}{\cos \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (85)$$

TABLE 1.—COMPUTATIONS FOR THE LOCATION OF THE NEUTRAL POINT, AND CO-ORDINATES OF THE ARCH POINTS, FOR ARCH NO. 1.

Point.	Δx.	x.	$\frac{dy}{dx}$.	$\cos \alpha$.	$\cos^5 \alpha$.	$y_c = \frac{x^2}{2k}$.	$y_c \cos^5 \alpha$.	y.
1	7.0	66.5	0.950	0.725	0.2005	31.60	6.33	−24.60
2	7.0	59.5	0.850	0.762	0.256	25.30	6.47	−18.30
3	7.0	52.5	0.750	0.800	0.328	19.68	6.45	−12.68
4	7.0	45.5	0.650	0.838	0.413	14.80	6.11	− 7.80
5	7.0	38.5	0.550	0.876	0.518	10.60	5.49	− 3.60
6	7.0	31.5	0.450	0.912	0.632	7.07	4.46	− 0.07
7	7.0	24.5	0.350	0.944	0.748	4.28	3.20	3.72
8	7.0	17.5	0.250	0.970	0.856	2.18	1.87	4.82
9	7.0	10.5	0.150	0.989	0.945	0.78	0.74	6.22
10	7.0	3.5	0.050	0.999	0.997	0.09	0.09	6.91
Σ	....	....	.....	.....	5.893	....	41.21	....

Taking into consideration that the thickness of the rib at any section is,

$$2c = 2c_0 \sec^2 \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (86)$$

Equations (84), (85), and (86) may be combined to give the relation,

$$\frac{z'}{c} = \frac{y \cos^3 \alpha}{\left(\frac{H_{DL}}{H_{RS}} - \cos^2 \alpha\right) c_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots (87)$$

A study will now be made of the elastic angle changes. No plastic deformations will be taken into consideration in this analysis.

The total angle change, from one end of the arch to the other, is represented by,

$$\sum \frac{d\phi_E}{ds} \Delta s = \sum k \left(\frac{M}{z' U}\right)^m \frac{\Delta x}{\cos \alpha}$$

$$= \frac{k}{b^m (c_0)^{1+2m}} \sum \frac{M^m}{\left(\frac{z'}{c} U'\right)^m} (\cos \alpha)^{1+4m} \Delta x \ldots (88)$$

in which, c is given by Equation (86), $\frac{z'}{c}$ by Equation (87), and,

$$U' = \frac{m}{1+m}\left[\left(\frac{u'}{c}+1\right)^{\frac{(1+m)}{m}} - \left(\frac{u'}{c}-1\right)^{\frac{(1+m)}{m}}\right]$$

which may be read from Fig. 10, showing the relation between $\frac{z'}{c}$ and U'.

Similarly, the horizontal movement of one springing relative to the other will be,

$$\Delta l = \sum \frac{d\,\phi_E}{d\,s}\, y_s\, \Delta s = \frac{k}{b^m\,(c_0)^{(1+2m)}} \sum \frac{M^m}{\left(\frac{z'}{c}\,U'\right)^m} (\cos\alpha)^{1+4m}\, y_s\, \Delta x \ldots (89)$$

in which, y_s is the vertical distance from a line joining the springings to an arch point.

Assume that the rib-shortening thrust, H_{RS}, acts through the neutral point corresponding to the ordinary theory of elasticity. Table 2 indicates the computations for the eccentricity of the thrust line, the angle changes, and the relative horizontal movement of the springings, for the ratio, $\frac{H_{DL}}{H_{RS}} = 25$ and $m = 1$. The relations obtained from the proposed theory are used in making the analysis. For the special case in which m is unity, Equations (88) and (89), respectively, become,

$$\sum \frac{d\,\phi_E}{d\,s}\, \Delta s = \frac{k}{b\,(c_0)^3} \sum \frac{M}{\frac{z'}{c}\,U'} \cos^5\alpha\, \Delta x \ldots\ldots\ldots (90)$$

and,

$$\Delta l = \frac{k}{b\,(c_0)^3} \sum \frac{M}{\frac{z'}{c}\,U'} \cos^5\alpha\, y_s\, \Delta x \ldots\ldots\ldots\ldots (91)$$

The elastic rib-shortening has not been included in Δl in either Equation (89) or Equation (91). For convenience, the reciprocal of the factors, $\frac{k}{b^m\,(c_0)^{(1+2m)}}$, has been called f in Tables 2 and 3. From Table 2,

$$\Delta l = -\frac{183}{144} \times 10^{-9}\,(961.2 - 356.7) \times 7 \times 2 \times H_{RS} = 0.00001075\, H_{RS}$$

Table 3 gives the angle changes and relative movement of the springings for the ratio, $\frac{H_{DL}}{H_{RS}} = 25$ and $m = 1.3$. Thus, Equation (88) and Equation (89), respectively, take the forms,

$$\sum \frac{d\,\phi_E}{d\,s}\, \Delta s = \frac{k}{b^{1.3}\,c_0^{3.6}}\, 10^{-0.2} \sum \left(\frac{M}{\frac{z'}{c}\,U'}\right)^{1.3} (\cos\alpha)^6 \times (10\cos\alpha)^{0.2}\, \Delta x \ldots (92)$$

$$\Delta l = \frac{k}{b^{1.3}\,c_0^{3.6}}\, 10^{-0.2} \sum \left(\frac{M}{\frac{z'}{c}\,U'}\right)^{1.3} (\cos\alpha)^6 \times (10\cos\alpha)^{0.2}\, y_s\, \Delta x \ldots (93)$$

The computations have been carried out on the assumption that the neutral point, considering the modulus of elasticity a variable, coincides with the neutral point in which $m = 1$. Using the results of Table 3,

$$\Delta l = \frac{183}{(144)^{1.3}} \times 10^{-9}\,(4\,043 - 1\,465)\, 10^{-0.2} \times 7 \times 2 \times (H_{RS})^{1.3}$$
$$= 0.00000648\, (H_{RS})^{1.3}$$

TABLE 2.—COMPUTATIONS FOR THE ECCENTRICITY OF THE THRUST LINE, ANGLE CHANGES, AND RELATIVE HORIZONTAL MOVEMENT OF SPRINGING. ARCH UNIFORMLY LOADED AND RIB-SHORTENING DUE TO ALL CAUSES.

$$\left(m = 1;\ \frac{H_{DL}}{H_{RS}} = 25.\right)$$

Point.	$\cos \alpha$. (1)	y. (2)	$\cos^2 \alpha$. (3)	$\cos^3 \alpha$. (4)	$\cos^5 \alpha$. (5)	$y \cos^3 \alpha$. (6)	$\frac{H_{DL}}{H_{RS}} - \cos^2 \alpha$. (7)	$\frac{z'}{c}$. (8)	$\frac{1}{\frac{z'}{c} U'}$. (9)	$f \frac{d\phi}{ds}$ = Column (2) × Column (5) × Column (9). (10)		y_s. (11)	$f \frac{d\phi}{ds} y_s$ = Column (10) × Column (11). (12)	
1	0.725	−24.60	0.526	0.381	0.2005	−9.37	24.474	−0.3830	1.5	+7.28	.....	3.40	+ 25.1	.....
2	0.762	−18.30	0.580	0.442	0.256	−8.08	24.420	−0.3340	1.5	+7.03	.....	9.70	+ 68.2	.....
3	0.800	−12.68	0.640	0.512	0.328	−6.48	24.360	−0.2665	1.5	+6.23	.....	15.32	+ 95.5	.....
4	0.838	− 7.80	0.702	0.588	0.413	−4.58	24.298	−0.1890	1.5	+4.83	.....	20.20	+ 97.7	.....
5	0.876	− 3.60	0.768	0.673	0.518	−2.39	24.232	−0.0985	1.5	+2.80	.....	24.40	+ 68.3	.....
6	0.912	− 0.07	0.832	0.759	0.632	−0.05	24.178	−0.0219	1.5	+0.07	.....	27.93	+ 1.9	.....
7	0.944	+ 2.72	0.890	0.840	0.748	+2.29	24.110	+0.0950	1.5	.....	− 3.06	30.72	.....	− 94.0
8	0.970	+ 4.82	0.940	0.912	0.856	+4.40	24.060	+0.1830	1.5	.....	− 6.18	32.82	.....	−203.2
9	0.989	+ 6.22	0.978	0.967	0.945	+6.01	24.021	+0.2500	1.5	.....	− 8.82	34.22	.....	−302.0
10	0.999	+ 6.91	0.998	0.997	0.995	+0.89	24.002	+0.2870	1.5	.....	−10.38	34.91	.....	−362.0
Σ	.....	.....	.....	.....	.....	.....	.....	.....	...	+28.34	−28.44	.....	+356.7	−961.2

TABLE 3.—COMPUTATIONS FOR THE ANGLE CHANGES AND THE RELATIVE HORIZONTAL MOVEMENT OF THE SPRINGINGS. ARCH UNIFORMLY LOADED AND RIB-SHORTENING DUE TO ALL CAUSES.

$$\left(m = 1.3;\ \frac{H_{DL}}{H_{RS}} = 25.\right)$$

Point.	y. (1)	10 cos α. (2)	log Column (2). (3)	0.2 × Column (3). (4)	(10 cos α)$^{0.2}$. (5)	cos$^6\alpha$. (6)	Column (5) × Column (6). (7)	$\frac{z'}{c}$. (8)	$\frac{1}{U'}$. (9)	$\frac{M}{\frac{z'}{c}U'}$. (10)	log Column (10). (11)	Column (11) × 1.3. (12)	$\frac{M^{1.3}}{\left(\frac{z'}{c}U'\right)^{1.3}}$. (13)	$f\frac{d\phi}{ds}$ = Column (13) × Column (7). (14)		y_s. (15)	$f\frac{d\phi}{ds}y_s$. (16)		$\frac{u'}{c}$. (17)	$f\frac{d\xi}{ds}$ = Column (14) × Column (17) / cos α. (18)
1	−24.60	7.25	0.860	0.1720	1.482	0.145	0.215	−0.384	0.680	43.5	1.638	2.130	135.0	+29.0	.....	3.40	+ 98	.....	0.70	28.0
2	−18.30	7.62	0.882	0.1764	1.491	0.195	0.291	−0.334	0.605	33.2	1.521	1.980	95.0	+27.7	.....	9.70	+ 268	.....	0.85	30.9
3	−12.68	8.00	0.903	0.1806	1.515	0.262	0.397	−0.267	0.507	24.0	1.380	1.796	62.5	+24.8	.....	15.32	+ 380	.....	1.03	32.0
4	− 7.80	8.38	0.923	0.1846	1.529	0.346	0.529	−0.189	0.390	16.1	1.207	1.569	37.1	+19.7	.....	20.20	+ 396	.....	1.41	33.2
5	− 3.60	8.76	0.942	0.1882	1.542	0.453	0.698	−0.099	0.260	9.42	0.974	1.266	18.5	+12.9	.....	24.40	+ 315	.....	2.52	37.0
6	− 0.07	9.12	0.960	0.1920	1.553	0.576	0.895	−0.022	0.117	0.37	.....	.....	0.3	+ 0.3	.....	27.93	+ 8	.....	8.20	38.0
7	+ 2.72	9.44	0.975	0.1950	1.567	0.707	1.108	+0.095	0.235	6.88	0.837	1.088	12.3	.....	−13.6	30.72	.....	− 418	2.71	38.9
8	+ 4.82	9.70	0.987	0.1974	1.575	0.831	1.309	+0.183	0.373	9.85	0.993	1.291	19.6	.....	−25.7	32.82	.....	− 845	1.47	39.1
9	+ 6.22	9.89	0.995	0.1990	1.581	0.936	1.480	+0.250	0.480	11.85	1.074	1.398	25.0	.....	−37.0	34.22	.....	−1268	1.10	41.2
10	+ 6.91	9.99	0.999	0.1998	1.584	0.994	1.575	+0.281	0.532	12.80	1.107	1.439	27.5	.....	−43.3	34.91	.....	−1512	1.00	43.2
Σ	.....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	+114.4	−119.6	.....	+1465	−4043	....	361.5

It is realized, of course, from the experimental data on the elastic properties of concrete that a variable relation of stress to elastic strain in which m is as high as 1.3 is probably never obtained in practice. However, an extreme value of m has been chosen in order that the results may show characteristic trends more prominently. Also, since a value of p of 1.3 in the proposed theory of plasticity is probably about correct for concrete 3 or 4 months old, this value makes a convenient starting point for an analysis in which the plastic properties of the material are considered.

A comparison of Tables 2 and 3 reveals some interesting phenomena. Both sets of computations were made on arches of identical dimensions and loadings, but of materials having very different elastic properties. In the first case (Table 2), Hooke's law was assumed to be true. This condition could have been solved by the ordinary theory of elasticity and the resulting differential angle changes, except for slight errors in precision, would have been identical with those obtained by an application of the proposed theory of elasticity, in which $m = 1$.

In both cases the rib-shortening thrust was considered to act in the same relative location. Although the rate of angle change at corresponding sections in the two arches differs greatly, the total angle change over the arch ring in both cases is practically equal to zero. However, change in span due to the rib-shortening thrust in the two cases varies, that for a constant modulus of elasticity being much less than that for a variable relation of stress to strain. For example, assume a uniformly distributed load to be placed on the arch of such a magnitude that the average compression at the crown is 200 lb. per sq. in. The rib-shortening thrust will be, $H_{RS} = 200 \times 12 \times 24 \div 25 = 2\,305$ lb. Hence, for $m = 1$, $\Delta l = 0.00001075 \times 2\,305 = 0.0248$ ft., and for $m = 1.3$, $\Delta l = 0.00000648 \times (2\,305)^{1.3} = 0.153$ ft.

As a result, assuming the coefficient of expansion in both arch materials to be constant and equal to each other, one might conclude that if the rib-shortening in the first case was due entirely to a temperature change of 36°, in the second case (a variable, E, being assumed) the temperature change which would cause the same stressed condition would be, $\Delta t^\circ = 36^\circ \times \frac{0.153}{0.0248} = 222^\circ$. This comparison, however, is not entirely fair, because if one were solving an arch problem by the theory of elasticity he would probably choose a value of E which would correspond to the secant modulus at the allowable working stresses, thus resulting in a different comparison. The foregoing comparison is based on the initial tangent modulus in both cases considered. Nevertheless, this analysis does indicate that temperature stresses computed by the ordinary theory of elasticity are incorrect.

Analysis of Elastic Arch Subjected to a Uniformly Distributed Dead Load and Dead Load Rib-Shortening

Consider Arch No. 1 to be uniformly loaded. If there were no rib-shortening effects, the line of pressure would coincide with the axis of the rib and there would be no stresses due to flexure. However, the rib of the arch shortens under the load and, as a result, flexural stresses arise.

Neglecting all but elastic deformations, which is the case obtained at the instant of striking the forms of a newly constructed arch, the condition that there shall be no horizontal movement of the abutments is defined by,

$$\sum k \left(\frac{M}{z'U}\right)^m y \frac{\Delta x}{\cos \alpha} + \sum k \left(\frac{M}{z'U}\right)^m u' \Delta x = 0 \quad \ldots\ldots (94)$$

Since the moments are due entirely to the rib-shortening thrust, H_{RS},

$$M = H_{RS}\, y$$

y being the distance from the line of action of the rib-shortening thrust to the arch point considered. Equation (94) may thus be transformed into,

$$(H_{RS})^m k \sum \left(\frac{y}{z'U}\right)^m y \frac{\Delta x}{\cos \alpha} + (H_{RS})^m k \sum \left(\frac{y}{z'U}\right)^m u' \Delta x = 0 \quad \ldots (95)$$

An examination of Equation (95) indicates that the eccentricity of the line of thrust at any point is independent of the intensity of the thrust, for, by dividing the equation by $(H_{RS})^m$, it will be observed that every factor is independent of the stress in the arch. Hence, it may be concluded that H_{RS} varies directly as the intensity of the dead load thrust, regardless of the values assigned to the parameters which define the properties of the material.

The problem now resolves itself into finding a ratio between the rib-shortening thrust and the dead load thrust which will give an eccentricity, z', satisfying Equation (95).

Solution for $m = 1$.—When $m = 1$, which is merely the case of the ordinary theory of elasticity, the problem is most easily solved by transforming Equation (95) into the form,

$$H_{RS} \sum k \frac{y^2}{I} \frac{\Delta x}{\cos \alpha} + \sum k \frac{N}{A} \Delta x = 0 \quad \ldots\ldots\ldots\ldots (96)$$

but assuming absolute values for force vector,

$$N = - H_{RS} \cos \alpha + \frac{H_{DL}}{\cos \alpha}$$

Substituting this value into Equation (96), and re-arranging the terms,

$$\frac{H_{RS}}{H_{DL}} = \frac{\sum \frac{\Delta x}{A \cos \alpha}}{\sum \frac{y^2}{I} \frac{\Delta x}{\cos \alpha} + \sum \frac{\cos \alpha}{A} \Delta x} \quad \ldots\ldots\ldots\ldots (97)$$

Table 4 has been computed on the basis of Parabolic Arch No. 1, in which,

$$\left.\begin{aligned} d &= d_0 \sec^2 \alpha \\ \text{hence,} \quad I &= \frac{1}{12} b\, d_0^3 \sec^6 \alpha \\ \text{and,} \quad A &= b\, d_0 \sec^2 \alpha \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (98)$$

Thus, when Equations (98) are substituted into Equation (97), and making $d_0 = 2$,

$$\frac{H_{RS}}{H_{DL}} = \frac{\Sigma \cos \alpha\, \Delta x}{3 \Sigma y^2 \cos^5 \alpha\, \Delta x + \Sigma \cos^3 \alpha\, \Delta x} \quad \ldots\ldots\ldots\ldots (99)$$

From Table 4,

$$\frac{H_{RS}}{H_{DL}} = \frac{8.815}{3 \times 401.1 + 7.071} = \frac{1}{137.3}$$

The eccentricity of the thrust line at each arch point is given in Table 5.

TABLE 4.—COMPUTATION OF RATIO OF HORIZONTAL THRUSTS FOR RIB-SHORTENING AND UNIFORMLY DISTRIBUTED LOAD ON ARCH NO. 1.

($m = 1$; $d_0 = 2.$)

Point.	Δx.	y.	$\cos \alpha$.	$\cos^3 \alpha$.	$\cos^5 \alpha$.	y^2.	$y^2 \cos^5 \alpha$.
1	7	−24.60	0.725	0.381	0.201	605.0	121.4
2	7	−18.30	0.762	0.442	0.256	335.0	85.8
3	7	−12.68	0.800	0.512	0.328	160.5	52.7
4	7	− 7.80	0.838	0.588	0.413	60.8	25.1
5	7	− 3.60	0.876	0.673	0.518	12.96	6.7
6	7	− 0.07	0.912	0.759	0.632	0.005	0.0
7	7	+ 2.72	0.944	0.840	0.748	7.40	5.5
8	7	4.82	0.970	0.912	0.856	23.2	19.9
9	7	6.22	0.989	0.967	0.945	38.6	36.5
10	7	6.91	0.999	0.997	0.995	47.8	47.5
Σ	..	.....	8.815	7.071	.....	.....	401.1

Solution for $m = 1.3$.—When the value of m is other than unity, a solution becomes more cumbersome. For this case, the most convenient form of Equation (95) seems to be,

$$(H_{RS})^m \sum \frac{k}{b^m c^{1+2m}} \frac{y^m}{\left(\frac{z'}{c} U'\right)^m} y_s \frac{\Delta x}{\cos \alpha} + (H_{RS})^m \sum \frac{k}{b^m c^{2m}} \frac{y^m}{\left(\frac{z'}{c} U'\right)^m} \frac{u'}{c} \Delta x = 0 \quad \ldots\ldots\ldots\ldots (100)$$

This relation takes the following form for both Arch No. 1 and Arch No. 2,

$$(H_{RS})^m \frac{k}{b^m (c_0)^{2m}} 10^{-0.2} \left[\frac{1}{c_0} \sum \frac{y^m}{\left(\frac{z'}{c} U'\right)^m} y_s (10 \cos \alpha)^{0.2} (\cos \alpha)^6 \Delta x + \sum \frac{y^m}{\left(\frac{z'}{c} U'\right)^m} \frac{u'}{c} (10 \cos \alpha)^{0.2} (\cos \alpha)^5 \Delta x \right] = 0 \quad \ldots\ldots\ldots (101)$$

The requirement that the abutments shall not rotate is satisfied when,

$$(H_{RS})^m \frac{k}{b^m (c_0)^{1+2m}} 10^{-0.2} \sum \frac{y^m}{\left(\frac{z'}{c} U'\right)^m} (10 \cos \alpha)^{0.2} (\cos \alpha)^6 \Delta x = 0 \quad \ldots (102)$$

To obtain the correct line of thrust now resolves itself into finding the value of the ratio, $\frac{H_{RS}}{H_{DL}}$, and the location of the line of action of H_{RS}, which will satisfy Equations (101) and (102).

Computations for Arch No. 1, in which $m = 1.3$, are shown on Table 6. The calculations are made on the assumption that the rib-shortening thrust is the same as that found for $m = 1$. It will be observed that the sum of the angle changes are practically equal to zero. However, considerable relative horizontal movement of the abutments is indicated. Consequently, one might conclude, the rib-shortening thrust lies in relatively the same line of action for the cases of constant E and variable E, but that the magnitude of this thrust is the smaller in the case of the variable modulus. The line of action for the rib-shortening thrust, however, lies slightly higher for the case of the variable modulus, this being evidenced by the fact that the angle change in the arch rib portion above the assumed line of action of the rib-shortening thrust is slightly greater than the sum of the angle changes in the portion below. (See Table 6, Column (11).)

TABLE 5.—COMPUTATION FOR ECCENTRICITY, $\frac{z'}{c}$, DUE TO UNIFORMLY DISTRIBUTED LOAD ON ARCH NO. 1.

$$\left(c_0 = 1;\ \frac{H_{DL}}{H_{RS}} = 137.3.\right)$$

Point.	y. (1)	$\cos \alpha$. (2)	$\cos^2 \alpha$. (3)	$\cos^3 \alpha$. (4)	$y \cos^3 \alpha$. (5)	$\frac{H_{DL}}{H_{SR}} - \cos^2 \alpha$. (6)	$\frac{z'}{c} = \frac{\text{Column (5)}}{\text{Column (6)}}$ (7)
1	−24.60	0.725	0.526	0.381	−9.37	136.8	0.0685
2	−18.30	0.762	0.580	0.442	−8.08	136.7	0.0592
3	−12.68	0.800	0.640	0.512	−6.48	136.7	0.0475
4	− 7.80	0.838	0.702	0.588	−4.58	136.6	0.0336
5	− 3.60	0.876	0.768	0.673	−2.39	136.5	0.0175
6	− 0.07	0.912	0.832	0.759	−0.05	136.5	0.00037
7	+ 2.72	0.944	0.890	0.840	+2.29	136.4	0.0168
8	+ 4.82	0.970	0.940	0.912	+4.40	136.4	0.0323
9	+ 6.22	0.989	0.978	0.967	+6.01	136.3	0.0441
10	+ 6.91	0.999	0.998	0.997	+6.89	136.3	0.0509

Analysis for Arch No. 2.—For Arch No. 2 the equation of the center line with reference to the crown as origin becomes, $y = \frac{x^2}{280}$ and $\tan \alpha = \frac{d\,y}{d\,x} = \frac{x}{140}$. Table 7 contains the computations for the location of the elastic neutral points. All distances are measured in feet. The constant factor, $\frac{3}{2}\ \frac{\Delta\ x}{b\,c_0^3}$, has been omitted from all partial elastic weights. Using the results from Table 7,

$$y' = \frac{41.63}{8.343} = 4.98.$$

The ratio of the horizontal thrusts for rib-shortening and uniformly distributed load, and the corresponding eccentricity of the thrust line are calculated in Table 8, Columns (1) to (12), inclusive, on the consideration that m is 1. From Table 8,

$$\frac{H_{RS}}{H_{DL}} = \frac{9.624}{3 \times 199.5 + 8.9} = \frac{1}{63.1}.$$

TABLE 6.—RELATIVE HORIZONTAL MOVEMENT OF CANTILEVERS TERMINATING AT NEUTRAL POINT ASSUMING COMPUTED LINE OF THRUST FOR ($m = 1$) IN ARCH NO. 1.

$$\left(m = 1.3;\ \text{assumed}\ \frac{H_{DL}}{H_{RS}} = 137.3.\right)$$

Point.	y_c	$\frac{z'}{c}$	$\frac{1}{U''}$	$\frac{1}{\frac{z'}{c}U'}$	$\frac{1}{\left(\frac{z'}{c}U'\right)^{1.3}}$	$\cos a$.	$(10 \cos a)^2$.	$\cos^5 a$.	$\cos^6 a$.	$y^{1.3}$	$f\frac{d\phi}{ds}$ Column (5) × Column (7) × Column (9) × Column (10)	Column (11) × y_c	$\frac{u'}{c}$	Column (5) × Column (7) × Column (8) × Column (10) × Column (13)
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	—24.60	0.0685	0.186	2.72	3.67	0.725	1.485	0.201	0.145	64.6	51.0	1255	3.97	267
2	—18.30	0.0592	0.163	2.75	3.72	0.762	1.500	0.256	0.195	43.8	47.7	862	4.60	289
3	—12.68	0.0475	0.137	2.88	3.96	0.800	1.515	0.328	0.262	27.3	43.0	545	5.60	301
4	— 7.80	0.0336	0.103	3.08	4.32	0.838	1.530	0.413	0.346	14.50	33.2	259	7.90	313
5	— 3.60	0.0175	0.058	3.28	4.68	0.876	1.542	0.518	0.453	5.35	17.5	62	14.2	285
6	— 0.07	0.00037	0.002	4.0	6.0	0.912	1.556	0.632	0.576	.....	....	....	.....	295
7	+ 2.72	0.0168	0.057	3.42	4.95	0.944	1.558	0.748	0.707	3.65	....	54	14.9	312
8	+ 4.82	0.0323	0.101	3.12	4.38	0.970	1.572	0.856	0.831	7.72	....	214	8.30	380
9	+ 6.22	0.0441	0.130	2.95	4.07	0.989	1.580	0.945	0.936	10.73	....	401	6.23	405
10	+ 6.91	0.0509	0.146	2.88	3.96	0.999	1.59	0.995	0.994	12.35	....	545	5.38	417
Σ	.....	.....	.....	.....	.....	.....	....	.....	.....	.....	192.5	—206.2	4297	.....	3264

Columns (13), (14), and (15) have been added to compute the change in span length due to rib-shortening. The distances from the axis of the arch to the center of rotation of the section, which are given as $\frac{u'}{c}$ in Column (14), are taken from Fig. 9 for the corresponding values of $\frac{z'}{c}$ computed in Column (11). A check is thus obtained on the rib-shortening thrust by comparing the lengthening of the arch due to angle changes (the summation of Column (7)) and the shortening of the arch due to rib-shortening (the summation of Column (15)).

TABLE 7.—COMPUTATIONS FOR THE LOCATION OF THE NEUTRAL POINT, AND CO-ORDINATES OF THE ARCH POINTS FOR ARCH NO. 2.

Point.	Δx.	x.	$\frac{dy}{dx}$.	$\cos \alpha$.	$\cos^5 \alpha$.	$y_c = \frac{x^2}{2k}$.	$y_c \cos^5 \alpha$.	y.
1	7.0	66.3	0.475	0.903	0.600	15.80	9.48	—10.82
2	7.0	59.5	0.425	0.920	0.658	12.65	8.32	— 7.67
3	7.0	52.5	0.375	0.936	0.719	9.84	7.07	— 4.86
4	7.0	45.5	0.325	0.951	0.780	7.40	5.77	— 2.42
5	7.0	38.5	0.275	0.964	0.830	5.30	4.40	— .32
6	7.0	31.5	0.225	0.976	0.887	3.54	3.14	1.44
7	7.0	24.5	0.175	0.985	0.927	2.14	1.98	2.84
8	7.0	17.5	0.125	0.992	0.961	1.09	1.05	3.89
9	7.0	10.5	0.075	0.997	0.983	0.39	0.38	4.59
10	7.0	3.5	0.025	0.9997	0.998	0.04	0.04	4.94
Σ	...	....	.....	.....	8.343	.....	41.63	.....

Computations for the relative displacements at the ends of the imaginary cantilevers connected to the arch springings and terminating at the neutral point, are given in Table 9 for the equilibrium polygon satisfying the ordinary theory of elasticity. The elasticity of the arch material is considered to satisfy the exponential stress-strain equation when $m = 1.3$. In this, as in Arch No. 1, the sum of the differential angle changes is practically equal to zero, although the lengthening due to angle changes (the summation of Column (12)) is greater than the shortening due to deformations along the axis of the rib (the summation of Column (14)).

In Table 10, calculations for angle changes and rib-shortening deformations were made considering that $\frac{H_{DL}}{H_{RS}} = 81.8$. It will be noticed that, for this ratio of the horizontal thrusts due to dead load and rib-shortening, respectively, the sum of the angle changes is practically equal to zero, and the change in span of the arch due to rib-shortening (summation of Column (15)) and the change in span due to angle changes (summation of Column (13)) are nearly equal in magnitude. Thus, since the line of thrust satisfies the conditions of statics and the geometric requirements imposed, it may be concluded that the line of thrust on which the computations in Table 10 are based, is correct.

TABLE 8.—COMPUTATION OF RATIO OF HORIZONTAL THRUSTS FOR RIB-SHORTENING AND UNIFORMLY DISTRIBUTED LOAD AND THE CORRESPONDING ECCENTRICITY OF THE LINE OF THRUST FOR ARCH NO. 2.

($m = 1$; $2\,c_0 = d_0 = 2$.)

Point.	Δx.	y.	$\cos \alpha$.	$\cos^3 \alpha$.	$\cos^5 \alpha$.	y^2.	$y^2 \cos^5 \alpha$.	$\cos^2 \alpha$.	$y \cos^3 \alpha$.	$\frac{H_{DL}}{H_{RS}} - \cos^2 \alpha$.	$\frac{z'}{c} = \frac{\text{Column (9)}}{\text{Column (10)}}$.	$z' = \frac{\text{Column (11)}}{\text{Column (8)}}$.	$\cos^4 \alpha$.	$\frac{u'}{c}$.	Column (2) × Column (13) × Column (14).
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	7	−10.82	0.903	0.736	0.600	117.1	70.26	0.815	−7.96	62.3	0.1278	0.1569	0.664	2.62	18.8
2	7	− 7.67	0.920	0.779	0.658	58.8	38.70	0.846	−5.97	62.3	0.0958	0.1132	0.716	3.50	19.1
3	7	− 4.86	0.936	0.820	0.719	23.6	16.98	0.876	−3.99	62.2	0.0641	0.0732	0.769	5.18	19.3
4	7	− 2.42	0.951	0.880	0.780	5.85	4.56	0.905	−2.08	62.2	0.0334	0.0369	0.819	9.90	19.6
5	7	− 0.32	0.964	0.895	0.830	0.10	0.08	0.928	−0.29	62.2	0.0047	0.0051	0.862	.....	19.9*
6	7	+ 1.44	0.976	0.929	0.887	2.06	1.83	0.952	1.34	62.1	0.0216	0.0227	0.905	15.50	20.2
7	7	2.84	0.985	0.956	0.927	8.07	7.48	0.970	2.72	62.1	0.0438	0.0452	0.941	7.65	20.4
8	7	3.89	0.992	0.976	0.961	15.13	14.54	0.988	3.80	62.1	0.0612	0.0621	0.970	5.43	20.5
9	7	4.59	0.997	0.991	0.983	21.1	20.75	0.994	4.55	62.1	0.0733	0.0737	0.988	4.52	20.5
10	7	4.94	0.9997	0.999	0.998	24.4	24.35	0.999	4.94	62.1	0.0795	0.0796	0.999	4.17	20.6
Σ	.	.....	9.624	8.941	.....	.....	199.53	.....	.....	....	.....	.....	.....	.....	198.9

* Interpolated.

TABLE 9.—COMPUTATIONS FOR RELATIVE DISPLACEMENT AT ENDS OF CANTILEVERS TERMINATING AT NEUTRAL POINT FOR EQUILIBRIUM POLYGON SATISFYING ELASTIC THEORY FOR ARCH NO. 2.

$$\left(m = 1.3;\ \text{assumed}\ \frac{H_{DL}}{H_{RS}} = 63.1.\right)$$

Point.	y. (1)	$\frac{z'}{c}$. (2)	$\frac{1}{U'}$. (3)	$\frac{1}{\frac{z'}{c}U'}$. (4)	$\frac{1}{\left(\frac{z'}{c}U'\right)^{1.3}}$. (5)	$\cos\alpha$. (6)	$(10\cos\alpha)^{0.2}$. (7)	$\cos^5\alpha$. (8)	$\cos^6\alpha$. (9)	$y^{1.3}$. (10)	$\frac{d\phi}{ds}$ = Column (5) × Column (7) × Column (9) × Column (10). (11)		$\frac{d\phi}{ds}\times y$. (12)	$\frac{u'}{c}$. (13)	Column (5) × Column (7) × Column (8) × Column (10) × Column (13). (14)
1	−10.82	0.1278	0.294	2.30	2.95	0.903	1.558	0.600	0.542	22.20	−55.1	....	595	2.03	124
2	− 7.67	0.0958	0.240	2.50	3.30	0.920	1.560	0.658	0.606	14.20	−44.3	....	340	2.63	126
3	− 4.86	0.0641	0.176	2.74	3.72	0.936	1.565	0.719	0.671	7.80	−30.5	....	148	4.04	132
4	− 2.42	0.0334	0.103	3.08	4.32	0.951	1.570	0.780	0.741	3.15	−15.9	....	39	7.85	131
5	− 0.32	0.0047	0.017	3.7	5.5	0.964	1.57	0.830	0.800	0.23	− 1.6	....	1	....	127*
6	1.44	0.0216	0.071	3.29	4.69	0.976	1.57	0.887	0.865	1.61	....	10.2	15	11.7	123
7	2.84	0.0438	0.127	2.90	3.99	0.985	1.58	0.937	0.923	3.88	....	22.7	64	6.05	139
8	3.89	0.0612	0.169	2.76	3.74	0.992	1.58	0.961	0.953	5.85	....	33.0	130	4.21	140
9	4.59	0.0733	0.195	2.66	3.56	0.997	1.582	0.983	0.980	7.25	....	40.0	184	3.50	141
10	4.94	0.0795	0.208	2.62	3.50	0.9997	1.586	0.998	0.998	7.97	....	44.0	218	3.18	141
Σ	.....	.....	.....	....	....	.....	.....	.....	.....	....	−147.4	149.9	1 734	....	1 324

*Interpolated.

TABLE 10.—COMPUTATION FOR ECCENTRICITY, $\frac{z'}{c}$, DUE TO RIB-SHORTENING WITH UNIFORMLY DISTRIBUTED LOAD ON ARCH No. 2.

$$\left(m = 1.3;\ \frac{H_{DL}}{H_{RS}} = 81.8.\right)$$

Point.	y.	cos α.	$y \cos^2 \alpha$.	$\frac{z'}{c}$.*	$\frac{1}{U'}$.	$\frac{1}{\left(\frac{z'}{c} U'\right)}$.	$\frac{1}{\left(\frac{z'}{c} U'\right)}$ 1.3.	(10 cos α)0.2.	cos^3 α.	cos^5 α.	y^1.3.	$f \frac{d\phi}{ds}$ = Column (7) × Column (8) × Column (10) × Column (11).	$f \frac{d\phi}{ds} y$.	$\frac{w}{c}$	Column (7) × Column (8) × Column (9) × Column (11) × Column (14).	cos^2 α.	$\frac{z'}{c}$ = Column (4) / Column (16).
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	—10.82	0.903	—7.96	—0.0985	0.243	2.47	3.23	1.55	0.600	0.542	22.20	—60.2	651	2.54	169	0.815	—0.1210
2	— 7.67	0.920	—5.97	—0.0788	0.195	2.64	3.52	1.56	0.658	0.606	14.20	—47.3	363	3.45	177	0.846	—0.0872
3	— 4.86	0.966	—3.99	—0.0493	0.143	2.90	3.99	1.56	0.719	0.671	7.80	—32.6	158	5.40	188	0.876	—0.0562
4	— 2.42	0.951	—2.08	—0.0256	0.082	3.19	4.52	1.57	0.780	0.741	3.15	—16.5	40	9.75	170	0.905	—0.0284
5	— 0.32	0.964	—0.288	—0.0035	0.014	4.0	6.0	1.57	0.830	0.800	0.23	— 1.7	1	....	165†	0.929	—0.0038
6	1.44	0.976	1.34	+0.0166	0.057	3.43	4.95	1.57	0.887	0.865	1.61	.....	16	14.90	165	0.952	+0.0174
7	2.84	0.985	2.72	0.0337	0.103	3.06	4.27	1.58	0.927	0.923	3.88	.....	69	7.80	189	0.970	0.0348
8	3.89	0.992	3.80	0.0470	0.137	2.92	4.02	1.58	0.961	0.958	5.85	.....	138	5.69	202	0.986	0.0477
9	4.59	0.997	4.55	0.0563	0.158	2.79	3.80	1.58	0.983	0.980	7.25	.....	196	4.70	202	0.994	0.0567
10	4.94	0.9997	4.94	0.0611	0.170	2.78	3.78	1.58	0.998	0.998	7.97	.....	235	4.23	202	0.999	0.0611
Σ	.....	.....	.....	.....	.....	....	....	....	.....	.....	.....	—158.3	1867	.....	1829	.....	.....

* Since in Equation (87) the value, cos^2 α, causes only an inappreciable variation in $\frac{z'}{c}$, $\frac{H_{DL}}{H_{RS}} - \cos^2 \alpha$ was assumed to be constant and equal to 80.9.

† Interpolated.

Both Tables 8 and 10 indicate the eccentricity of the line of thrust at various points along the rib for Arch No. 2 under uniformly distributed vertical loading. For the case in which strain varies more rapidly than stress, that is, when $m = 1.3$ (Table 10), the line of thrust lies closer to the arch axis than when stress is considered proportional to strain, that is, when $m = 1.0$ (Table 8).

Studies similar to these have been made for non-uniform loading over an arch. In these investigations, which are not here included, it was found that, neglecting rib-shortening, the lines of thrust were practically identical for different values of m. Rib-shortening influences become greater for low than for high values of m.

A Study of the Effect of Shrinkage

Arch No. 2 will be considered to be loaded with a uniformly distributed vertical load, and subject to plastic, elastic, and shrinkage deformations. The shrinkage will be assumed to be a definite function of the time, and the plastic flow will be considered to be a function of the time and stress intensity. In order to make the problem as simple as practicable it will be considered that no shrinkage causing internal stresses occurs until the centering supporting the newly constructed arch is removed. At that moment, all deformations are due to elastic distortions, and plastic and shrinkage deformations begin simultaneously. The problem, then, is to find the location of the line of thrust in the arch at the end of any time interval after striking the centering.

Consider the properties of the material to be defined by the exponential equations with the following parameters: $m = 1.3$; $p = 1.3$; $k = 183 \times 10^{-9}$; $K = 17 \times 10^{-9}$; and $q = 0.40$.

The eccentricity of the line of thrust at the crown when the structure is in equilibrium elastically before any plastic or shrinkage deformations have taken place, has been found to be $0.0611 \times 12 = 0.733$ in. With the stated conditions, since $m = p$ and neglecting shrinkage, there could be no movement of the line of thrust due to plastic flow.

Assume that the shrinkage is equal to,

$$\gamma = \eta\, t^q \qquad (103)$$

in which

$$\eta = 145 \times 10^{-7}$$

This would result in a unit shrinkage of about 0.0002 in two years.

The equations, stipulating, respectively, that the relative horizontal displacement of the springings and the total angle change in the arch rib shall be zero, may be written for Arch No. 2, as follows:

$$\frac{10^{-9.2}\,(183 + 17\,t^{0.4})}{(144)^{1.3}}\left[\sum\left(\frac{M}{\frac{z'}{c}\,U'}\right)^{1.3}(\cos\alpha)^6\,(10\cos\alpha)^{0.2}\,y_s\,\Delta x + \sum\left(\frac{M}{\frac{z'}{c}\,U'}\right)^{1.3}(\cos\alpha)^5(10\cos\alpha)^{0.2}\,\frac{v'}{c}\,\Delta x\right] - 140 \times 145 \times 10^{-7}\,t^{0.4} = 0. \qquad (104)$$

TABLE 11.—COMPUTATIONS FOR RELATIVE MOVEMENT OF SPRINGINGS OF ARCH No. 2, WHEN UNIFORMLY LOADED AND

$$m = 1.3; \frac{H_{DL}}{H_{RS}} = 71.8$$

Point.	y.	$\frac{z'}{c}$.*	$\frac{1}{U}$.	$\frac{1}{\left(\frac{z'}{c}U\right)}$.	$\frac{1}{\left(\frac{z'}{c}U\right)^{1.3}}$.	$\cos\alpha$.	$(10\cos\alpha)^{0.2}$.	$\cos^5\alpha$.	$\cos^6\alpha$.	$y^{1.3}$.	$f\frac{d\phi}{dx}$ = Column (5) × Column (7) × Column (9) × Column (10).	$\frac{u'}{c}$.	Column (5) × Column (7) × Column (8) × Column (10) × Column (12).	y_s.	$fy_s\frac{d\phi}{dx}$.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	−10.82	0.1125	0.268	2.38	3.08	0.903	1.55	0.600	0.542	22.20	− 57.5	2.26	144	1.70	98.7
2	− 7.67	0.0842	0.218	2.59	3.44	0.920	1.56	0.658	0.606	14.20	− 46.3	2.98	150	4.85	224.5
3	− 4.86	0.0563	0.158	2.80	3.81	0.936	1.56	0.719	0.671	7.80	− 31.1	4.67	156	7.66	238.2
4	− 2.42	0.0294	0.094	3.19	4.51	0.951	1.57	0.780	0.741	3.15	− 16.4	8.70	152	10.10	165.6
5	− 0.32	0.0041	0.016	3.9	5.7	0.964	1.57	0.830	0.800	0.23	− 1.6	....	148†	12.20	19.5
6	1.44	0.0189	0.063	3.33	4.78	0.976	1.57	0.887	0.865	1.61	.....	12.90	143	13.96	.....
7	2.84	0.0384	0.116	3.03	4.22	0.985	1.58	0.927	0.923	3.88	.....	6.85	165	15.36	.....
8	3.89	0.0536	0.152	2.84	3.88	0.992	1.58	0.961	0.953	5.85	.....	4.98	173	16.41	.....
9	4.59	0.0642	0.176	2.74	3.72	0.997	1.58	0.983	0.980	7.25	.....	4.08	173	17.11	.....
10	4.94	0.0696	0.188	2.70	3.63	0.9997	1.58	0.998	0.998	7.97	.....	3.78	173	17.46	.....
Σ	.....	.....	.....	....	....	.....	....	.....	.....	....	−152.9	....	1577	.....	746.5

															2587.7
											156.0				

*Assumed that $\frac{H_{DL}}{H_{RS}} - \cos^2\alpha = 70.9$ since the greatest variation from this value is 0.1 and thereby causes an inappreciable effect.

† Interpolated.

$$\frac{10^{-9.2}\,(183 + 17\,t^{0.4})}{(144)^{1.3}}\left[\sum\left(\frac{M}{\frac{z'}{c}\,U'}\right)^{1.3}(\cos\alpha)^6\,(10\cos\alpha)^{0.2}\,\Delta x\right] = 0 \quad \ldots\ldots (105)$$

Before any effects of shrinkage or plastic flow have taken place, the ratio of the dead load horizontal thrust to the rib-shortening horizontal thrust will be 81.8. This calculation has been indicated in Table 10. Since shrinkage takes place, one might well presuppose that the rib-shortening thrust will become greater. It will be considered, therefore, that the ratio, $\frac{H_{DL}}{H_{RS}}$, is equal to 71.8. Computations for the summations are shown in Table 11. By substituting these summations in Equation (105), and collecting the terms, the following numerical relation is obtained:

$$t = \left[-\frac{698}{64.7 - \frac{2\,090\,000}{H_{RS}{}^{1.3}}}\right]^{2.5} \quad \ldots\ldots (106)$$

Assuming that the uniformly distributed load causes an average thrust of 350 lb. per sq. in. at the crown,

$$H_{RS}{}^{1.3} = \left(350 \times 12 \times 24 \times \frac{1}{71.8}\right)^{1.3} = (1\,405)^{1.3} = 12\,400$$

and,

$$t = \left[-\frac{698}{64.7 - \frac{2\,090\,000}{12\,400}}\right]^{2.5} = 118 \text{ days}$$

The elastic change in length of span resulting from this rib-shortening thrust would be,

$$\Delta l = \frac{183}{(144)^{1.3}} \times 10^{-9}\,(1\,841 - 1\,477)\,2 \times 7 \times 10^{-0.2}\,H_{RS}{}^{1.3} = 674 \times 10^{-9}\,H_{RS}{}^{1.3}$$

For a load producing a crown thrust of 350 lb. per sq. in., the change in span length would be, $\Delta l = 674 \times 10^{-9} \times 12\,400 = 0.00836$ ft., or 0.100 in., which is extremely small.

To take shrinkage into account, it has been considered that the crown eccentricity of the thrust line would be a total of $0.0696 \times 12 = 0.836$ in., or $0.836 - 0.733 = 0.103$ in. greater than the normal eccentricity when the structure is in elastic equilibrium. As a result, of course, when the time interval, t, is zero, a slight horizontal displacement of the abutments ($\Delta l =$ 0.1 in.) is assumed. A period of 118 days, then, is the time which it would take for this horizontal displacement to become zero, considering that both plastic flow and shrinkage follow the assumed laws and that the average crown thrust is 350 lb. per sq. in. Thus, the eccentricity of the thrust line would become somewhat greater.

Consider, now, the case in which the denominator in the second member of Equation (106) is zero. That is,

$$64.7 - \frac{2\,090\,000}{H_{RS}{}^{1.3}} = 0$$

or,

$$H_{RS} = \left(\frac{2\,090\,000}{64.7}\right)^{\frac{1}{1.3}} = 2\,920 \text{ lb.}$$

Then, the average dead load thrust at the crown will be,

$$\frac{H_{DL}}{12 \times 24} = \frac{2\,920 \times 71.8}{12 \times 24} = 728 \text{ lb. per sq. in.}$$

Thus, if the average thrust at the crown were 728 lb. per sq. in., or a total horizontal thrust of $728 \times 24 \times 12 = 210\,000$ lb., the rate of change of span length due to plastic deformation would be just equal and opposite to that due to shrinkage. As a result, t becomes infinite.

If the properties of the concrete approximate those assumed for the analysis, the foregoing considerations make it difficult to conceive how shrinkage of the concrete can materially affect the stresses in the arch.

Deflection of Crown of Arches

General Equations.—Fig. 28 indicates a loaded arch, the line of thrust being indicated by a full line.

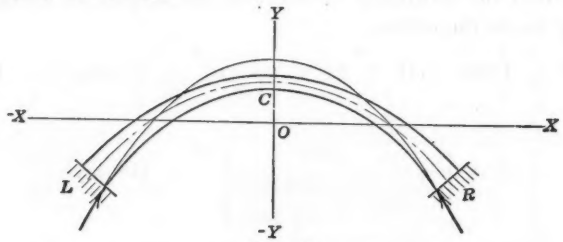

Fig. 28.

If the springing, L, does not rotate, the vertical deflection of the crown, C, must be,

$$\delta_c = \int_C^L \frac{d\,\phi}{d\,s} \frac{x\,d\,x}{\cos\alpha} + \int_C^L \frac{d\,\xi}{d\,s} \tan\alpha\, d\,x \ldots\ldots\ldots (107)$$

regardless of whether the deformations are elastic, plastic, or a combination of both. Replacing the definite integrals of Equation (107) by finite summations, and considering only elastic deformations, the deformations following the relations,

$$\frac{d\,\phi}{d\,s} = k\left(\frac{M}{z'\,U}\right)^m, \text{ and } \frac{d\,\xi}{d\,s} = k\,w'\left(\frac{M}{z'\,U}\right)^m$$

there results the following equation for the vertical deflection of the crown of the arch:

$$\delta_c = k\sum_C^L \left(\frac{M}{z'\,U}\right)^m \frac{x\,\Delta\,x}{\cos\alpha} + k\sum_C^L w'\left(\frac{M}{z'\,U}\right)^m \tan\alpha\,\Delta\,x \ldots\ldots (108)$$

If Hooke's law is considered to hold, Equation (108) may be rewritten in the form,

$$\delta_c = \sum_C^L \frac{M\,x\,\Delta\,x}{E\,I\cos\alpha} + \sum_C^L \frac{T\tan\alpha\,\Delta\,x}{E\,A} \ldots\ldots\ldots\ldots (109)$$

In case both elastic and plastic deformations are considered, Equation (107) could be written as,

$$\delta_c = k \sum_C^L \left(\frac{M}{z'\,U}\right)^m \frac{x\,\Delta\,x}{\cos\,\alpha} + K\,t^q \sum_C^L \left(\frac{M}{z'\,V}\right)^p \frac{x\,\Delta\,x}{\cos\,\alpha}$$

$$+ k \sum_C^L w' \left(\frac{M}{z'\,U}\right)^m \tan\,\alpha\,\Delta\,x + K\,t^q \sum_C^L w' \left(\frac{M}{z'\,V}\right)^p \tan\,\alpha\,\Delta\,x \ldots (110)$$

If $n = p = m$, then $W = V = U$, and Equation (110) simplifies to,

$$\delta_c = (k + K\,t^q) \left[\sum_C^L \left(\frac{M}{z'\,W}\right)^n \frac{x\,\Delta\,x}{\cos\,\alpha} + \sum_C^L \left(\frac{M}{z'\,W}\right)^n w' \tan\,\alpha\,\Delta\,x\right]. \quad (111)$$

Vertical Deflection of Crown of Arch No. 2 When Subjected to a Uniformly Distributed Vertical Load.—Arch No. 2 is considered to be loaded with a uniform load of w lb. per ft. of length. Tabular computations, not here included, based on the foregoing equations, were carried out to study the crown deflection due to rib-shortening effects when m is equal to 1 and 1.3, respectively.

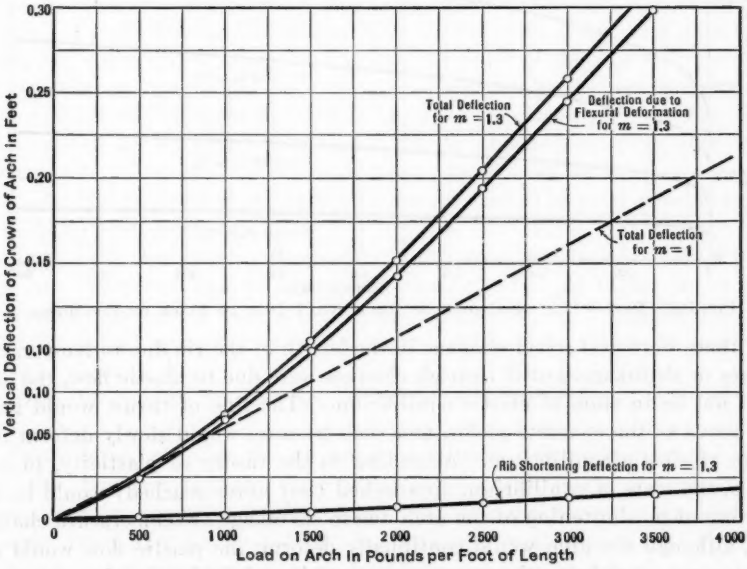

FIG. 29.—VERTICAL DEFLECTION OF CROWN OF ARCH NO. 2.

In order to have a basis for comparison of the deflection of the crown of an arch composed of a concrete following Hooke's law with one in which the exponential equation governs the relation of stress to strain, the parameters, E and k, have been chosen so that the deflection will be the same for both arches when the average stress at the crown section is 300 lb. per sq. in.

In Fig. 29 are shown the crown deflections of the arch for various magnitudes of the uniformly distributed load. For the case of a variable modulus

of elasticity, several curves have been drawn; only the total deflection has been plotted for the arch in which material of construction is assumed to have a constant modulus of elasticity.

Crown Deflection, Considering Plastic Flow.—As a simple example of the anticipated effect of plastic flow on the crown deflection of an arch, Arch No. 1 is considered to be subjected to a uniformly distributed load and to be undergoing rib-shortening as well as temperature change. The following numerical values are used: $m = p = 1.3$; $k = 613 \times 10^{-10}$; $K = 56.7 \times 10^{-10}$; and $q = 40$.

It is considered that the ratio, $\frac{H_{DL}}{H_{RS}}$, is equal to 25. In this case, the rib-shortening force, H_{RS}, is due to all causes, including temperature change, shrinkage, and rib-shortening. The location of the resulting line of thrust is indicated in Table 2, together with computations for angle changes in the arch rib.

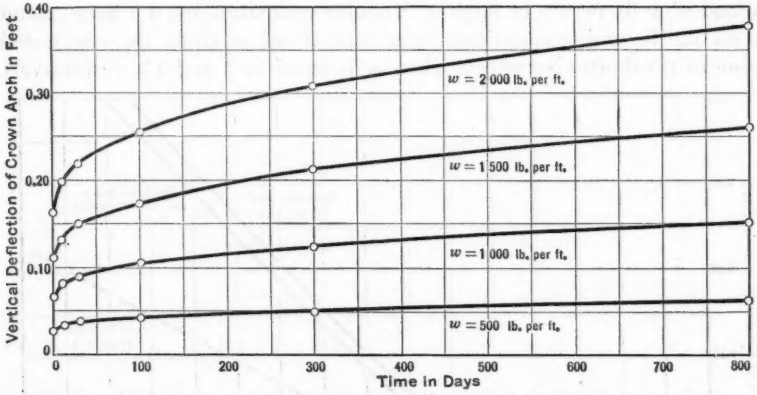

FIG. 30.—DEFLECTION OF CROWN OF ARCH NO. 1 DUE TO FLOW OF CONCRETE.

If there were no further changes in the length of the rib due to temperature changes or shrinkage, but if instead, changes were due to plastic flow, the arch would not be in state of elastic equilibrium. The line of thrust would gradually move as the concrete yields, and the structure would slowly deform into a state of elastic equilibrium. According to the theory of plasticity, in case $m = p$, the state of equilibrium approached (but never reached) would be the condition of no shortening of the arch due to shrinkage or temperature change. Thus, although the arch would continually deform, the plastic flow would act in such a way as to bring the structure back to its original state of equilibrium.

However, for simplification, and as a hypothetical case, the structure is supposedly held in elastic equilibrium by assuming that the temperature drops at such a rate that the tendency of the plastic flow to bring the structure to its original state of equilibrium is just offset by the reverse action caused by temperature change. As a result, the structure changes in shape although the relative location of the line of thrust and the stress distribution remains unchanged. In Fig. 30 are curves based on computations for various uniform load distributions over the arch.

Conclusion

Inasmuch as the investigation presented in this paper is based on an impartial interpretation of physical facts regarding the behavior of concrete under sustained load, the writer feels justified in pointing out a few conclusions which appear unquestionable as a result of his study:

First.—It is incorrect to assume that in general plastic flow in statically indeterminate concrete structures causes a redistribution of bending moments tending to relieve the overstressed parts of the structure.

Second.—If there is an external disturbance encountered by a statically indeterminate concrete structure, such as the settlement of a support, the plastic flow causes a gradual redistribution of stresses such that the structure approaches (but never reaches) its original state of equilibrium (no settlement of support). When no external disturbances occur, plastic flow will cause progressive changes in the deflections and displacements of points on the structure, but will cause little or no change in the moment distribution. In this connection, temperature changes as well as rib-shortening and shrinkage in arches are included as external disturbances.

Third.—Provided disturbances due to reinforcing steel are not important, it is evident that striking the centering of a fixed arch at an early date after pouring the concrete has many advantages. In spite of the fact that the total rib-shortening resulting will be greater, the stresses occasioned thereby and in consequence of the immediate movement of the abutments will be less. The greatest effects on the stress distribution due to rib-shortening and movement of abutments will occur when the centering is allowed to support the arch far beyond the time required, although for this case the total rib-shortening will probably be a minimum.

Fourth.—As far as all experimental data on concrete examined by the writer are concerned, plastic flow would cause the line of thrust in an arch to approach the center axis more closely. However, it is conceivable that plastic flow might cause the line of thrust to be pulled away from the axis; this will occur when $m > p$.

Fifth.—In making an analyses of the rib-shortening stresses in an arch it is of vital importance to distinguish between shrinkage, temperature change, and elastic deformation on the one hand, and plastic deformation on the other.

Sixth.—The fact that the ratio of stress to elastic strain does not follow Hooke's law for concrete usually causes but little difference in moment distribution from that computed by the ordinary elastic theory. For rib-shortening and temperature changes the difference may be greatly on the side of safety.

Seventh.—Stresses computed by the ordinary elastic theory for temperature changes are likely to be greatly in error, but on the side of safety for high temperature variations.

Eighth.—From the analyses of the crown deflections of an arch it appears well to sound a warning against misinterpretation of test data. The writer believes, especially for newly constructed arches, that although the crown deflections measured in stressed arches may check computations made by the ordinary theory of elasticity in many cases, the location of the line of thrust

may be very different from that indicated by working back from the measured deflection by the usual theory.

The new theories presented in this paper are not intended to replace the ordinary method of analyses used heretofore, but are developed with the idea of supplementing the older theory of elasticity. For important statically indeterminate structures, such as long-span, fixed concrete arch bridges, an analysis seems incomplete without an investigation by more accurate methods than is provided by the usual theory in which stress is assumed to be proportional to strain and deformation due to flow is neglected. For smaller structures the usual procedure for stress analysis is probably satisfactory. The proposed methods serve further as tools for engineers in research and help to develop more accurate conceptions regarding the behavior of concrete structures.

Much more experimental work, especially of a quantitative nature, is desirable and necessary in order to establish definitely the laws of plastic flow for concrete of various mixes and ages.

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PAPERS AND DISCUSSIONS

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EXPERIMENTS ON DISCHARGE OVER SPILLWAYS AND MODELS, KEOKUK DAM

Discussion*

By Floyd A. Nagler and Albion Davis, Members, Am. Soc. C. E.†

Floyd A. Nagler,‡ and Albion Davis,§ Members, Am. Soc. C. E. (by letter).‖—The writers are deeply appreciative of the constructive comment made by the various discussers. Apparently, little remains to be said in closing the discussion. It is, indeed, gratifying to note the close agreement between the coefficients obtained on the Keokuk model and those presented by Mr. Ott,¶ resulting from experiments at the Research Institute at Munich. The writers' estimate that their experiments were reliable within 2% was not based upon the laboratory verification by means of the Pitot tube alone; it was a judgment based upon the general reliability of the current-meter methods and ratings used in the field, together with the various checks which were afforded by other measurements and the model tests.

Additional data were requested** relative to the behavior of various types of current meters in streams with varying degrees of angularity, both from the left and the right. The average corrections applied in the paper were based on angular calibrations made at the U. S. Bureau of Standards. However, these were subsequently checked in an extensive investigation†† at the laboratory of the University of Iowa, made in conjunction with the Bureau of Public Roads, U. S. Department of Agriculture.‡‡

* Discussion of the paper by Floyd A. Nagler and Albion Davis, Members, Am. Soc. C. E., continued from October, 1929, *Proceedings*.

† Authors' closure.

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‖ Received by the Secretary, November 21, 1929.

¶ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1879.

** *Loc. cit.*, August, 1929, Papers and Discussions, p. 1658.

†† *Loc. cit.*, December, 1929, Papers and Discussions, p. 2611.

‡‡ "Effect of Turbulence on the Registration of Current Meters", by David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2611.

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PAPERS AND DISCUSSIONS

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USE OF WATER ON FEDERAL IRRIGATION PROJECTS

Discussion*

BY MESSRS. RALPH L. PARSHALL AND E. B. DEBLER†

RALPH L. PARSHALL,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—In presenting this paper with its official records of the use of water on Federal irrigation projects, the author has made available data of great value to irrigation engineers. Because of its importance, however, it is thought that he should have gone more into detail in explaining the underlying causes for the large percentage of the total diversion that is returned to the parent stream, since a study of the data presented might lead to the belief that project waste and canal losses are much greater than should have been expected; and to those not thoroughly familiar with irrigation requirements, it might appear that such wastage showed inefficient operation.

In practically all the irrigated areas in the West, the matter of sand and silt carried into both main canals and laterals is of grave concern from the standpoint of operation. It is the usual practice to provide one or more wastegates at suitable points along the main canals and, through these openings, to flush out the accumulated deposits. The water used for sluicing purposes is, in many cases, a large proportion of the total diversion. Thus, Table 25‖ indicates marked wastage for the Rio Grande and Yuma Projects. These items are explained by the author as necessary in supplying other areas at lower elevations and providing water for hydro-electric power. Such expenditures in water might not be looked upon as waste, since they are put to a useful purpose. The use of the word, "waste", is not in accordance with the general meaning, which is "loss without equivalent gain." In this restricted sense, the writer is of the opinion that Table 25 might be misleading to some in pre-

* Discussion of the paper by E. B. Debler, M. Am. Soc. C. E., continued from December, 1929, *Proceedings*.

† Author's closure.

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§ Received by the Secretary, November 30, 1929.

‖ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, pp. 777-778.

senting these large percentages. With the irrigation canals diverting from the Arkansas River in Colorado, for example, it is the usual practice to sluice occasionally for the purpose of scouring out the accumulated deposits. In some cases this is done each day for a period of 1 hour, an actual waste of water to the users amounting to about 4 per cent. In no case would this waste exceed 10 per cent. The water used in sluicing constitutes a direct loss to the canal. Another fruitful source of waste occurs through leaky farm and lateral head-gates, as well as from poor structures, such as flumes, siphons, or other artificial channels.

The canal and lateral losses appear to be large, especially for some of the projects having large percentages of lined canal or pipe lines. In the case of the Umatilla Project, the miles of pipe and lined section together approximate the total miles of canals and laterals; yet the losses are recorded as being not greatly different in percentage from those on other projects with a much smaller proportion of lined channel and with similar soil conditions. Concrete-lined canals are not entirely immune from seepage losses.

The measurement of water from the standpoint of duty requires a fair degree of accuracy. Because of the great variety of conditions and limitations imposed in the measurement, the author assumes that the actual amounts measured are somewhat less than 10% smaller than those stated in the tables. Unless the conditions are extremely adverse, the writer believes the deviation ought not to exceed 5%, provided suitable devices are used in the measurement. The calibrated rating flume is usually a poor means of securing dependable measurements. The filling and scouring of the channel and the accumulation of aquatic growth, all tend materially to affect the accuracy of this type of device. The rating flume, if only occasionally checked for performance, may introduce considerable error in the final result. In some cases constant attention to check gaugings is necessary to determine the daily rate of change in the calibration. The necessary hydrographic expense of keeping abreast of such conditions may not be warranted because of allowable limits assumed in the management or administration of the water supply. Because of aquatic growth some Colorado canals equipped with the ordinary rectangular rating flume have changed between June and September as much as 75% in the calibration. Infrequent check gaugings, in such cases, would involve considerable error in estimating discharge. In many parts of the irrigated West, where the value of water is becoming increasingly greater, better and more dependable measurements of flow are much needed.

The use of the standard weir for the measurement of farm deliveries is generally unsatisfactory. For correct measurements the weir must be provided with standard distances, crest to floor and side of weir to bank or wall. Weirs without end contractions eliminate one of these sources of error. Almost without exception, in the case of the farmer's weir, the pool or basin up stream is found to be filled with sand or silt nearly or actually to the crest line. This tends to a greater delivery than is apparent from the indicated head or depth over the crest. The farmer is scientific enough to know that this defect is advantageous to him, and is, therefore, not overly ambitious to maintain standard conditions.

Submerged weirs and orifices are also subject to a varying degree of accuracy; however, in such cases, the errors will be compensating. Ordinarily, in farm practice, no instrument record is maintained of the variation in the upper and lower heads, where a change in either or both results in a corresponding change in the flow. Thus, in using a submerged device, the chance of error is greater than in the single head measurement. A standard weir submerged to about 70% will reduce the rate of discharge approximately 35 per cent. In endeavoring to obviate the double-head submerged flow over weirs or through orifices, it is found that the improved Venturi flume, when operating at about 70% submerged flow, will be discharging according to the free-flow law and thus only involves a single head. In this respect it is believed that this type of device is superior to other commonly used methods of measurement.

In the study of the duty of water, there is a question as to whether or not it is proper to add the total precipitation over a period of months to the quantity or depth of water applied as irrigation to represent the acre-feet per acre application to the crop. When storms occur in moderation and the soil can absorb the precipitation, such depth of rainfall can be legitimately charged. For an intense downpour, when the top soil is dry, it has been observed that the surface becomes packed or crusted, due to the impact, thus causing a considerable run-off. It is further known that, for showers of 0.1 to 0.2 in., the penetration is not sufficient to be of any benefit from the standpoint of adding moisture to the soil; in fact, quite the contrary is true. Observations on soil moisture after a moderate shower show a depletion of moisture instead of an increase. The question as to what amount of the total rainfall can be justly accredited to useful soil moisture has not been determined, the soil itself being an important factor. It is logical, therefore, to assume that to give full weight to the total rainfall is misleading. The question deserves thorough study.

E. B. DEBLER,* M. AM. SOC. C. E. (by letter).†—Interesting data are presented by Mr. Monroe‡ on a class of projects which is quite common in California, but is rarely found outside that State. California has large areas characterized by deep finely textured soils of unusual water-holding capacity. The winter rains are stored in these soils, and, to a large degree, furnish the water needs in the succeeding growing season with a resulting high duty of irrigation water. Furthermore, these lands are in nearly every case adapted to fruit culture and so escape the need for the liberal water supply required for the growing alfalfa and similar crops of high water requirement which must necessarily be the backbone of irrigation in the inter-mountain States.

Mr. Houk graphically presents§ the relation of depth of water applied to the length of growing season. The most important feature controlling application of water is the mechanical texture of the soil and sub-soil. Soils of great porosity require heavy applications wherever general farming is practiced. On the Orland Irrigation Project, in California, annual applications of

* Hydr. Engr., U. S. Bureau of Reclamation, Denver, Colo.
† Received by the Secretary, November 16, 1929.
‡ *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1685.
§ *Loc. cit.*, p. 1688.

water vary from 2 to 6 ft. per acre, and in the Snake River Valley, in Idaho, from 2 to 10 ft. per acre. The lack of fertility which is so characteristic of most soils of extreme porosity precludes, in individual cases, the production of fruits in the one area and of sugar beets in the other, as these crops are seldom profitable except on fertile soils of considerable depth.

The effects of continual application of large quantities of water in the reduction of soil fertility has been mentioned by Mr. Carter* and has been noticed in many localities on the Federal projects. The only way of escape where this is the case appears to be the adoption of crops requiring intensive cultivation and permitting a comparatively heavy expenditure in the control of water so that application will not exceed greatly the water-holding capacity of the surface soils. This course, however, can be adopted only to a limited extent in that a very large part of the irrigated area is not adapted to the production of such crops by reason of unfavorable climate and marketing difficulties.

Mr. Houk has presented much data bearing on comsumptive use, a subject even more far-reaching than that of the delivery of water, to which the paper applies. It is hoped that the Irrigation Division of the Society will delve deeply into the matter of consumptive use in the near future, in view of approaching exhaustion of irrigation water resources on a number of streams. Where development is governed by water supply the undeveloped resources in the way of non-irrigated lands will lead to a study of the ways and means to reduce consumptive uses to a minimum for each unit of crops that can be produced profitably.

Mr. Carter suggests† that canal losses might well be estimated from exposed water surface. The U. S. Bureau of Reclamation conducted experiments for a number of years on constructed canals in order to ascertain the coefficients of loss, in feet per day, over the wetted area. The loss through soils was found to vary from amounts as low as 0.25 ft. to more than 1 ft. of depth per day. Over the same section of canal the losses were also found to vary greatly from one year to another. For a type of soil given the same description by different engineers, the range in losses reported varied between very wide limits. A part of the differences are due no doubt to variance in definitions of soils by different men. In order to obtain a uniform definition it would be necessary to make a mechanical analysis of the soils; but it is also well known that mechanical properties alone will not tell the entire story, since the nature and distribution of soil chemicals and humus are very important factors.

The depth to ground-water is important because, with a very high water-table there is a back pressure from the water-table to the canal. Where silty waters are carried, an impervious skin coat forms on the canal banks and the leakage is determined, not by the soils through which the canal passes, but by the thickness and character of the deposit of silt. Where it is customary to remove the silt from a canal at the end of each irrigation season there may be a very great variation in losses through the season. On the other hand, if a

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2691.
† *Loc. cit.*, p. 2692.

canal is originally built with excess capacity, it may be the practice in operation to leave a large part of the silt coat undisturbed.

Aside from interpreting the data obtained on operating systems, many assumptions are necessary in applying such formulas as might be evolved to the conditions that will obtain on a system proposed for construction. The combined result of all these features is that the observed losses cannot practicably be converted into formulas or coefficients that can be applied in designing a new canal system. On the whole, it has been found more practicable to make a direct estimate of the percentage of loss on new projects, with such percentage greatest at the end of the irrigation season. This, however, can be only considered a practical application of the theory that losses are largely dependent on the wetted areas which do not differ materially through the irrigation season. Except where conservation of water is of no interest, losses on the average project do not vary greatly from 35 to 40% of the waters diverted. Where a reduction in losses becomes desirable, either for the prevention of seepage or on account of limited stream supply in comparison with area, leading to the adoption of canal lining, the loss may be materially less than the percentages indicated.

Mr. Carter mentions* the desirability of a description of classification of soils on the various projects. This matter was given consideration, but it was found that nearly every one of the projects has soils and sub-soils ranging from heavy to loose and that two observers will seldom give the same classification for any one kind of soil. It would be difficult to convey to the reader an accurate picture of soil and sub-soil types. While soils are the most important feature in determining the water duty, it would also be very desirable to have full descriptions of irrigation methods, crops, liberality of water supply, etc. Impracticability of precise descriptions for these factors within the limits of a paper of moderate length, precludes their discussion.

Attention has been directed to the apparent large waste. Contrary to the understanding of Mr. Parshall,† the indicated data do not include sluicing waste at the point of diversion. On some projects stream flow far exceeds all irrigation requirements and excess waters are carried through a large part of the season for greater ease in operation, with the irrigator permitted to receive and reject water practically at will. A similar condition prevails on projects where stream flow is merely passed through project canals for the use of other areas. On projects where conservation of water and canal capacity are important, waste is not heavy except in years of surplus supply. The Boise Project, with an average waste of 2%, is of this type.

Mr. Parshall appears to have gained the impression that the arbitrary correction in recorded deliveries was made to offset estimated errors in measurement. This was, however, not the case. Measurements have not been disturbed. The increases were made to offset deliberate practice in the field of reporting less delivery than is actually made, in the interest of improved business relations between the farmer and the ditch-rider.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2692.

† See p. 117.

The magnitude of precipitation during the growing season has been found to be very material in fixing irrigation requirement. Along the eastern slope of the Rocky Mountains, growing season precipitation averages 10 in., while in the Great Basin, it averages 2 in. The latter, and possibly an equal part of the former, occurs largely in small showers without benefit to crops except as relative humidity is temporarily increased. The remainder of the larger amount materially reduces the irrigation requirement. In fact, on some projects, a little farther east, in the so-called "semi-arid" belt with normally 12 to 15 in. of rainfall in the growing season, irrigation is considered an adjunct to farming rather than the mainstay, and is so practiced.

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EXPERIMENTS TO DETERMINE RATE OF EVAPORATION FROM SATURATED SOILS AND RIVER-BED SANDS

Discussion*

By Carl Rohwer, Assoc. M. Am. Soc. C. E.

Carl Rohwer,† Assoc. M. Am. Soc. C. E. (by letter).‡—The evaporation from moist soils is an important factor in the application of irrigation water to crops and in the determination of the quantity of water stored in the soil, due to rainfall. The measurement of the evaporation presents some difficulty, but the method described by Mr. Parshall permits more accurate control of the ground-water level than is possible by other methods, and materially reduces the work of making observations. As explained by the author,§ the Mariotte control apparatus is affected by temperature, and care should be used in insulating the equipment as completely as possible.

The principal effect of temperature on the apparatus is in the expansion and contraction of the air in the chamber above the water surface, and in the change in the vapor pressure in the air with the change of temperature. Both effects increase as the temperature increases. The pressure of the air in a closed vessel is directly proportional to the absolute temperature, but the change in vapor pressure with the temperature does not follow so simple a law. The variation, however, has been determined experimentally and shows that there is an appreciable effect on the pressure, due to the change of vapor pressure. The relative magnitude of the two effects of temperature is not important. The only concern is the total effect of the two forces.

When the temperature increases in the Mariotte control apparatus, the pressure in the chamber, *D*, Fig. 1,§ increases and destroys the balance. As a result, water is driven from the water tank. This water usually raises the level of the water in the soil tank slightly, and if the increase in temperature

* Discussion of the paper by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., continued from October, 1929, *Proceedings*.

† Assoc. Irrig. Engr., U. S. Dept. of Agriculture, Fort Collins, Colo.

‡ Received by the Secretary, November 30, 1929.

§ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 844.

is great enough some of the water will drain out at K into C. In case the evaporation from the soil surface exceeds the rate at which the water is driven from the water tank, due to the increase in temperature, no error in the record will result. The only effect will be in decreasing the amount of air drawn through the tube, T, to maintain the balance. If the temperature drops, the pressure in D will decrease and, as pointed out by the author,* air should be drawn in through the tube, T, to re-establish the balance in the apparatus. Actually, however, the change in temperature is so gradual that the force is not sufficient to break the bubble from the lower end of the tube, T, and, as a result, water is drawn from the soil back into the tank. The result of this condition is that the indicated evaporation is too small.

The original 1928† records of the observations on the evaporation from saturated soils and river-bed sands show that there was a definite rise and fall of the water in the Mariotte tubes, due to the change in temperature. It was found that Tank No. 17 (Fig. 2),‡ which contained coarse sand and in which the water level was maintained at a depth of 1 ft. below the surface, lost no water during the season. There was, however, a pronounced rise and fall of the water in the Mariotte apparatus which, apparently, bears some relation to the rise and fall of the air temperature. This is shown graphically in Fig. 5, which is a plot of the air temperature at 7:00 A. M. and 7:00 P. M., taken from the meteorological records of the Colorado Experiment Station, and the gain and loss of the water levels in Tank No. 17, in centimeters, for the 12-hour intervals between the 7:00 P. M., 7:00 A. M., and 7:00 P. M. readings for each 24-hour period during the month of August, 1928. The plot indicates, in general, that the water rose in the Mariotte tank when the temperature decreased, and fell when the temperature increased; but the amount of the rise and fall was not proportional to the change in temperature. This is probably due to the fact that since the tanks were carefully insulated, the temperature in the tank did not follow the temperature of the air very closely.

For comparison, the record of the evaporation loss from a free-water surface under similar conditions, as shown by Tank No. 7 (Fig. 2), is also given. The plot shows that the loss due to evaporation from the free-water surface is many times the fluctuation in Tank No. 17 due to the change in temperature. If the comparison had been made on the basis of 24-hour intervals, the difference would have been several times greater because the loss from the free-water surface would be nearly doubled and the net change in Tank No. 17 would have been considerably less. The author has shown (Fig. 4)§ that the evaporation loss from coarse river sand and heavy adobe, Tanks Nos. 14 and 17 (Fig. 2), is very small when the water-table is 12 in. beneath the surface. Temperature probably has an appreciable effect on the indicated evaporation loss from these tanks for a 24-hour period, but by increasing the time interval it is possible to reduce the effect until the desired accuracy is obtained.

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 849.

† Unpublished records of the Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Agricultural Experiment Station.

‡ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 845.

§ *Loc. cit.*, p. 854.

Another feature of the Mariotte control apparatus which must be considered is the accuracy with which it controls the level of the water surface in the soil and in the water tanks. No accurate records are available as to the limits within which the apparatus controls the levels in the soil tanks, but the 7:00 A. M. and 7:00 P. M. readings on the hook-gauges in the tanks with the free-water surfaces (Tanks Nos. 7 and 8, Fig. 2) give an accurate measure of the limits within which the apparatus permits the water surface to vary. A study of these records for Tanks Nos. 7 and 8 for the 1928 season, shows that there is a variation between 0.02 and 0.04 in. in the elevation of the water surface between the 7:00 A. M. and 7:00 P. M readings, except during periods of rain when any variation is possible, depending on the amount of the rainfall. This, of course, would be corrected by the overflow from the outlet at *K* (Fig. 1), if sufficient time was allowed to elapse.

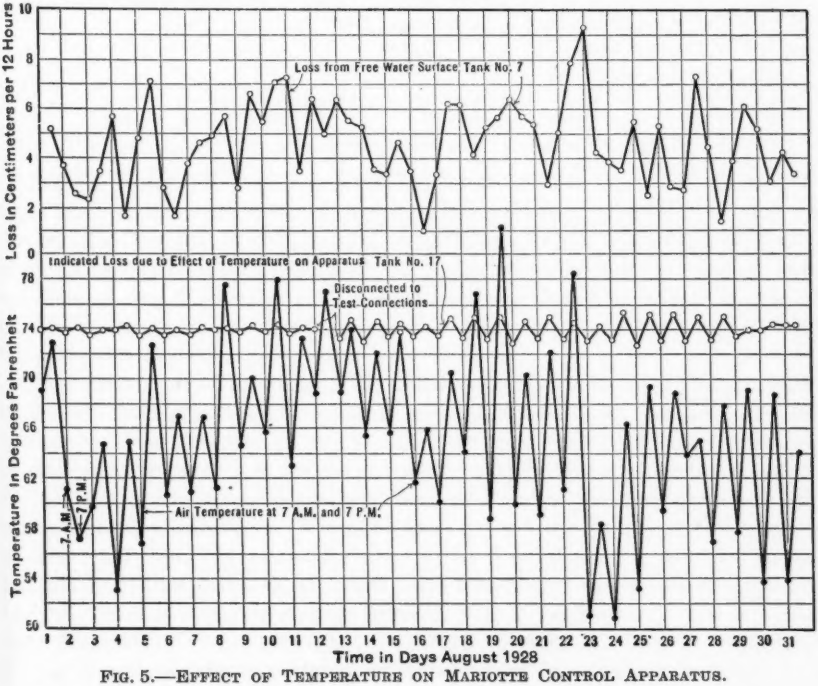
Loss in Centimeters per 12 Hours
Loss from Free Water Surface Tank No. 7
Indicated Loss due to Effect of Temperature on Apparatus Tank No. 17
Disconnected to Test Connections
Temperature in Degrees Fahrenheit
7 A.M.
7 P.M.
Air Temperature at 7 A.M. and 7 P.M.
Time in Days August 1928

FIG. 5.—EFFECT OF TEMPERATURE ON MARIOTTE CONTROL APPARATUS.

The Mariotte control apparatus gives reliable results in so far as the control of the water level and the measurement of the evaporation loss are concerned, but there are still some features of the design that need improving because of the difficulty in keeping all the connections air-tight. Even a slight leak permits enough air to leak into *D* (Fig. 1), so that the overflow from *K* will more than fill the receptacle, *B*. Experience with the tanks at Fort Collins, Colo., shows that most of the leaks occur in the connections to the glass gauge tubes. The writer believes that this condition could be corrected by introducing the tube, *T*, into the chamber, *D*, through a packed gland. This would permit using a smaller gauge glass which would be much easier to keep air-tight.

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WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS

Discussion*

By Ivan E. Houk, M. Am. Soc. C. E.

Ivan E. Houk,† M. Am. Soc. C. E. (by letter).‡—The results of the author's hydrologic studies are both interesting and valuable, especially in view of the fact that studies of deep rainfall penetration, or percolation, as it is more often termed, have seldom been made for irrigated sections of the country. His conclusion§ that "the water supply resulting from deep penetration of rainfall on the valley floors is of great economic importance", is true in the more humid States as well as in the semi-arid West.

In a general consideration of the disposal of rainfall it may be permissible to include deep penetration in lieu of seepage run-off and changes in ground storage.‖ Deep penetration would equal seepage run-off in cases where the ground-water surface has the same elevation, and the moisture stored in the surface soil is the same, at the beginning and end of the time interval under consideration, assuming no water to pass beyond the limits of the basin by underground flow. In many drainage areas throughout the country, including gently rolling and comparatively level topography as well as mountainous regions, ground-water, or seepage run-off, may be appreciably different from deep penetration as far as yearly, or shorter, periods of time are considered. The author shows the importance of seepage flow in mountain drainage areas in his treatment of rainfall and run-off in the Devil Canyon Basin.¶ Because of the importance of ground-water run-off it seems better, as a general rule, to consider the disposal of rainfall, as follows: (1) Flood, or storm-water, run-off; (2) ground-water, or seepage, run-off; (3) evaporation, including interception and transpiration losses, as well as evaporation from soil, water, snow,

* Discussion of the paper by A. L. Sonderegger, M. Am. Soc. C. E., continued from December, 1929, *Proceedings*.

† Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary November 26, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1139.

‖ *Loc. cit.*, p. 1140.

¶ *Loc. cit.*, p. 1155.

and ice surfaces; and (4) additions to ground storage, the last being positive or negative depending on whether the quantity of water in the ground has increased or decreased during the time interval considered.

A discussion of the general conditions that affect deep penetration might well include some reference to the capillary moisture in the surface soil. Obviously, if this has been greatly depleted by evaporation and transpiration processes at the time the rain begins, a large storage reservoir will exist in the root zone of the soil, and this reservoir will absorb large quantities of water before much deep penetration takes place, even though fairly uniform wetting of the soil may not occur to the full depth of the root zone. The importance of this factor is clearly shown by the author's discussion of the soil-moisture tests made on North Mesa, in the Murrietta-Temecula Area.* Furthermore, a consideration of capillary movements of soil moisture may be of some value in forming conclusions regarding the depth of uniform wetting of the soil. The author undoubtedly is correct in his conclusions† that "absorption is favored by the wedging action of roots," and that percolation tends to take place along well-defined ducts. However, it is quite likely that a fairly uniform wetting of the soil, to appreciable depths, if not to the full depth of the root zone, often takes place during the rainy season of the year, due to both capillary and gravity movements of soil moisture; and that in such cases the percolation of moisture through the minute passageways between soil particles is an important factor in conducting water to the well-defined ducts. Capillary movement of soil moisture may take place in any direction, horizontally, vertically upward, or inclined in either upward or downward directions, as well as vertically downward. Consequently, the vertical trickling streams at the ends of the suspended water-table, shown in Fig. 1,† will occur only when the geologic formations below the horizontal clay layers are relatively open, as, for instance, in the case of coarse sand or gravel deposits. In the case of tighter formations the water at the ends of the clay layers will move horizontally and diagonally beneath the clay by both capillary and gravity action. However, in any case, there will undoubtedly be some deficiency of moisture immediately below the clay.

In studying percolation of rainfall it is important to know how fast water can enter different soil formations; and how the rates of absorption vary for a given soil, using the term, absorption, to include all water that enters the ground, percolation water as well as the capillary moisture taken up in the unfilled pore spaces of the soil. A clay formation, after it becomes wet enough to close the surface cracks by swelling of the material, will absorb water at a relatively low rate, if at all.‡ On the other hand a deep open gravel, or a sod surface underlaid at a comparatively shallow depth by open sand or gravel, will absorb rainfall at a relatively high rate. Experiments on a small plat in the Miami Valley of Southwestern Ohio showed that blue-grass sod, on level ground, underlaid by sand and gravel deposits at a depth of about 2 ft., would absorb

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1148.

† *Loc. cit.*, p. 1141.

‡ "Water Penetration in the Gumbo Soils of the Belle Fourche Reclamation Project," by O. R. Mathews, U. S. Dept. of Agriculture, *Bulletin No. 447*, 1916, p. 4.

water continuously at a rate of 13.0 in. per hour.* Soil deposits having rates of absorption of practically all values between these limits probably exist in various parts of the country. The maximum rate of absorption for a given soil may be somewhat lower in winter than in summer, due to the lower soil temperatures, the friction coefficients being higher for the lower temperatures. This effect is probably more pronounced in the case of relatively tight formations, than in coarse sand or open gravel deposits. However, in the colder regions, the maximum rate of absorption during the winter months is probably increased by the loosening of the surface soil, due to alternate freezing and thawing, much more than it is decreased due to the higher friction.

The experimental rainfall and run-off investigations in the Miami Valley showed that the maximum rate of absorption for soils in that vicinity, under ordinary conditions of compactness, becomes practically constant within a relatively short time after the beginning of the storm; that is, as soon as the surface storage becomes filled.† The surface storage includes storage space in the upper few inches of the soil; storage space formed by slight depressions in ground-surface level; and storage space on the ground surface formed by forest litter, accumulations of leaves, blades of grass, weed stems, and other forms of vegetation, both dead and alive. During the late summer and fall months, when the ground was comparatively dry, and the cultivated surface soils compacted by earlier rainfalls, from 3 to 4 in. of rain in 1 or 2 days were required to fill the surface storage. Flood run-off usually occurred sometime before the rate of soil absorption became constant, the length of time from the beginning of the storm until flood run-off began naturally being greater for the less intense rates of precipitation. Except in the case of the sod plats, underlaid at shallow depths by open sand and gravel, a rate of rainfall of 1 in. per hour caused flood run-off to begin in about 30 min., even on the dryest of soils; whereas a rate of rainfall of 3.00 in. per hour invariably produced flood run-off in less than 5 min. Rates of flood run-off were directly proportional to the rates of rainfall in all cases, regardless of the quantity of moisture in the soil. However, the actual value of the relation varied with the moisture content. In other words, the relation between the rate of rainfall and the rate of run-off could be represented by a straight line equation in all cases, but the actual value of the equation varied with the moisture content. For saturated soil conditions the average rate of absorption was approximately inches per hour. Apparently, the loosening of the surface soil permitted the plowed fields, the rate of absorption was found to increase somewhat as the rate of rainfall increased, the relation being represented by the equation

$$p = 0.37\, r + 0.16 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, r and p are the rates of rainfall and absorption, respectively, both in inches per hour. Apparently, the loosening of the surface soil permitted the building up of a static head which increased with the rate of rainfall. A similar effect, but not so great, was found in the experiments on a hillside where the slope of the ground was about 18 ft. per 100 ft. Incidentally, one of the

* "Rainfall and Runoff in the Miami Valley," by Ivan E. Houk, Miami Conservancy Dist. Technical Repts., Pt. VIII, Dayton, Ohio, 1921, p. 116.

† *Loc. cit.*, p. 124.

principal conclusions reached as a result of the investigations was that variations in surface slope are of much less importance as affecting flood run-off than variations in vegetable cover, or in the condition of the surface soil. In the experiment on spaded soil, water was applied at a rate of 3.73 in. per hour for 22 min. before any run-off took place.

The Miami Valley experiments herein discussed were made primarily for the purpose of studying flood run-off during intense storms. Rainfall was produced artificially, by sprinkling with ordinary garden sprinkling cans, and the total depths applied during the different experiments varied from 6.00 to 17.83 in. Each experiment was made up of several separate runs and the average rates of application during the different runs varied from about 0.2 in. to about 4.0 in. per hour. Soil formations at the locations of the plates were compact yellow clay tills, alluvial deposits underlaid by glacial tills, and yellow sandy loam underlaid by open sand and gravel. Complete descriptions of plats, as well as of methods of experimentation, results, etc., are included in the reference cited.

The all-important factor as regards percolation of rainfall on a given drainage area is undoubtedly storm rainfall. However, this factor is made up of two parts: First, the rate of rainfall during the storm; and, second, the duration of the storm. For instance, a storm with a total rainfall of 6 in. in 2 days will cause much more percolation if the rain is distributed uniformly through the 48 hours than if the greater part of the total falls in a relatively short time, say, 4 or 6 hours. When the rate of rainfall exceeds the rate of absorption an increase in the rate of rainfall means a higher rate of storm run-off and a lower total depth of absorption for the same total rainfall. Of course, an increase in the duration of the storm period, with the same rate of rainfall, means an increase in the total depth of absorption regardless of whether or not the rate of rainfall exceeds the rate of absorption. If the storm occurs after the surface storage has been filled the rate of absorption may be practically constant, and, in that case, the increase in total depth of absorption is directly proportional to the increase in storm duration.

The rate of absorption as herein discussed may, or may not, be wholly a rate of percolation. In the case of deep, open, sand and gravel deposits the water entering the ground would be practically all percolation, from the beginning of the rain to the end of the storm period. Consequently, the rate of absorption would be the rate of percolation throughout the storm. However, in most soil formations, the rate of absorption at the beginning of the storm would be definitely greater than the rate at which deep percolation would be taking place, the exact amount of the excess depending on the dryness of the surface soil. In the case of a comparatively shallow surface soil, underlaid by open sand or gravel deposits, subjected to an intense storm at the end of an extended dry period, the rate of absorption would include very little deep percolation at the beginning of the rain, but would include more and more percolation as the storm continued, approaching the maximum rate as a limit by the time the surface storage became filled, or within a comparatively short time thereafter. However, in the case of a drainage area with relatively deep soil deposits, it

may, as the author states,* "require a very wet year to recharge a deeply depleted soil." In such cases the rates of absorption would include but small proportions of percolation during the entire year.

The author's treatment of seepage and storm run-off in the Devil Canyon drainage area is pertinent and logical.† However, the results, as shown on Fig. 8,‡ are rather surprising inasmuch as the seepage run-off is such a large proportion of the total. The reason for the comparatively small storm run-off must be the dense surface cover of brush and trees. Similar storm rainfall on more open mountain water-sheds with similar surface slopes and similar geological formation, would probably cause much greater proportions of storm run-off. In fact, the writer is inclined to believe that the retarding effect of the dense vegetal cover is much more important than the accelerating effect of the comparatively steep ground slopes; and that, consequently, cultivated valley floors in that vicinity, subjected to the same storm rainfall, would have greater proportions of storm run-off, regardless of whether or not the underlying soil formations were clay. In this connection it may be valuable to refer again to the rainfall and run-off investigations in the Miami Valley of Southwestern Ohio, where similar studies of seepage and storm run-off were made.§

TABLE 17.—COMPARISON OF AVERAGE ANNUAL STORM AND SEEPAGE RUN-OFF, IN MIAMI VALLEY FOR YEARS 1915 TO 1919, INCLUSIVE.

Drainage area.	Area, in square miles.	Average rainfall, in inches.	Total run-off, in inches.	Storm Run-Off.		Seepage Run-Off.	
				Inches.	Per-centage of total.	Inches.	Per-centage of total.
Miami River.....	2 525	37.8	12.67	8.57	67.6	4.10	32.3
Buck Creek	163	38.6	10.89	4.83	44.3	6.06	55.7
Mad River.....	652	39.5	13.78	7.34	53.3	6.44	46.7
Stillwater River..	600	37.5	13.58	10.70	78.8	2.88	21.2

Table 17 gives a comparison of the average annual storm and seepage run-off for four Miami Valley drainage areas, for 1915 to 1919, inclusive. For those areas the average annual storm run-off is seen to vary from 44.3 to 78.8% of the total run-off, and the average annual seepage run-off, from 21.2 to 55.7% of the total run-off. The drainage areas are more than 90% agricultural land and may be classed as gently rolling topography with elevations varying from about 1 100 ft. above mean sea level near the head-waters to about 700 ft. above mean sea level at the bottom of the river in Dayton. The reason for the lower proportions of storm run-off in the Buck Creek and Mad River drainage areas is that the surface soil in those water-sheds is relatively shallow and is underlaid by open sand and gravel deposits, thus permitting comparatively rapid rates of percolation. In the Pomperaug Basin, in Connecticut, the annual ground-water run-off, for a 3-year period, was found to be

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1143.
† *Loc. cit.*, p. 1155.
‡ *Loc. cit.*, p. 1156.
§ Miami Conservancy Dist. Technical Repts., Pt. VIII, 1921, p. 164.

8.76 in., or 42% of the total run-off, the remaining 58% being storm run-off.* The Pomperaug Basin is a rural agricultural area of about 89 sq. miles, made up of rather rugged uplands and extensive valley areas, the elevations varying from a minimum of 100 ft. to a maximum of 1 150 ft. above sea level. Surface soil deposits consist primarily of glacial drift, although some of the steeper slopes are practically bare bed-rock.

The author has secured some very interesting and valuable results in his exhaustive study of water supply from rainfall on valley floors. However, it is doubtful whether conclusions based thereon can be safely applied to other drainage areas, even in the immediate vicinity, without careful independent checking. This applies to conclusions regarding storm rainfall and flood run-off phenomena as well as to seepage run-off, percolation, and the resulting water supply. The geological, topographical, and meteorological conditions which affect rainfall, run-off, and percolation on different drainage areas are so many, so varied, and so variable that no intensive study of a single drainage area, or even of a group of drainage areas in the same locality, valuable as it may be in solving local problems, can furnish a satisfactory basis for general conclusions, or for the establishment of general laws universally applicable in studies of such phenomena.

* "A Study of Ground Water in the Pomperaug Basin, Connecticut, with Special Reference to Intake and Discharge," by Oscar Edward Meinzer and Norah Dowell Stears, U. S. Geological Survey, *Water Supply Paper 597-B*, p. 115.

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SILT TRANSPORTATION BY SACRAMENTO AND COLORADO RIVERS AND BY THE IMPERIAL CANAL

Discussion*

By Harry F. Blaney, Assoc. M. Am. Soc. C. E.

Harry F. Blaney,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper is of vital interest now (1929) since it is probable that large sums of money will be spent during the next ten years building structures to control and utilize the waters of the Colorado River. The life of storage reservoirs built will depend upon the quantity of silt transported by the Colorado River.

At various times since 1917 the writer has conducted silt investigations on the Colorado River and in Imperial Valley for the U. S. Department of Agriculture under the general supervision of Samuel Fortier, M. Am. Soc. C. E., and Mr. W. W. McLaughlin.

Under the immediate direction of the late C. E. Tait, M. Am. Soc. C. E., the writer assisted Mr. J. E. Peck in collecting a large percentage of the data shown in Tables 1§ and 5.‖ A detailed description of the field and laboratory methods and equipment used is given in a report¶ published by the U. S. Department of Agriculture.

The results of the mechanical analyses shown in Table 1, indicate that the character of the bed silt as regards size of particles, in the bed of the main canal, varies with the discharge of the Colorado River. Thus, during March, 1917, the mean flow of the river was 9 800 sec-ft., and the sieve analysis for March 14 of that year showed only 0.05% of the recently deposited silt in the bed of the Alamo Canal coarse enough to be retained on a No. 40 sieve. On July 15, 1917, eleven days after the passing of the peak of the spring flood

* Discussion of the paper by C. E. Grunsky, Past-President, Am. Soc. C. E., continued from November, 1929, *Proceedings*.

† Irrig. Engr., Div. of Agricultural Eng., U. S. Dept. of Agriculture, Los Angeles, Calif.

‡ Received by the Secretary, November 27, 1929.

§ *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1481.

‖ *Loc. cit.*, p. 1491.

¶ *Technical Bulletin No.* 67, "Silt in the Colorado River and Its Relation to Irrigation", by Samuel Fortier and Harry F. Blaney, 1928, pp. 13, 36, 37, 38, and 41.

carrying 143 000 sec-ft., the recent deposits in the bed of the Alamo Canal were much coarser in grain. At that time 12.09% was retained on a No. 20 sieve, 20.88% passed the No. 20, but was retained on the No. 40 sieve; 33.4% passed the No. 40 and was retained on the No. 60 sieve; while 7.28% passed a No. 80 sieve. These results indicate that bed silt is transported and that the high waters bring in most of the heavier silt.

Some appreciable time would be necessary, of course, for the heavier silt to travel down to the lower reaches of the canal system. If this fact is kept in mind and the July 15 analysis of the Hanlon Heading samples is disregarded, no great difference will be apparent between the character of the bed silt near the intake and that in the lower parts of the system.

In computing the volume of suspended silt carried by the Colorado River (see Tables 6,* 7† and 8‡), Mr. Grunsky assumed the dry weight of suspended silt as 100 lb. per cu. ft. Presumably, this is based on the data given in Table 1. The writer§ is of the opinion that this value is too high for "suspended silt," because the material in the samples analyzed in Table 1 was mostly sand or "bed silt" and included very little of the finer "suspended silt." Usually, the canal bottom was hard and compact and, at such times, difficult to sample. Published estimates indicate that the weight of 1 cu. ft. of suspended silt ranges from 33 to 93 lb. according to Mr. C. S. Howard.‖

That very little of the "suspended silt" was deposited in the beds of the larger Imperial Canals, is indicated by the data shown in Table 5. Mr. Tait, in referring to these data, stated:¶

"The percentages obtained from the seventeen other stations located on the system below the Hanlon Station show no appreciable reduction, thereby indicating that while great volumes of silt are removed from the canals by mechanical means each year, the amount deposited in the canals is a very small proportion of the total amount at the intake and that most of the silt entering the canals passes on to the lands irrigated."

The results of the mechanical analysis of "suspended silt" samples taken in the Colorado River at Yuma, Ariz., and the Main Imperial Canal at Hanlon Heading, Calif., by Mr. Tait and the writer, indicate that most of the suspended silt was too fine to be retained on a No. 300 sieve. Table 19 gives the results of some typical analyses of suspended silt. At the Yuma Station the total depth was 11 ft., the discharge was 23 500 sec-ft., and the mean river velocity was 4.82 ft. per sec. At Hanlon, the depth to the bottom was 10 ft., the discharge was 5 749 sec-ft., and the mean river velocity was 4.1 ft. per sec.

It will be noted from Table 19 that 73 to 94% of the silt carried in suspension by the Colorado River at Yuma and 77 to 95% in the Main Imperial Canal was fine enough to pass a No. 200 sieve. The difference between the two is small.

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1493.

† *Loc. cit.*, p. 1494.

‡ *Loc. cit.*, p. 1495.

§ *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 71.

‖ *Water Supply Paper 636-B*, U. S. Geological Survey, "Suspended Matter in Colorado River in 1925–28", by C. S. Howard, p. 27.

¶ Seventh Biennial Rept., Dept. of Eng., State of California, 1919–1920, p. 118.

A comparison of the data shown in Table 19 with the results in Table 1, indicates that the silt found in the beds of the Imperial Canals is considerably coarser than that carried in suspension at Yuma or Hanlon Heading, since only 1.7 to 34% of the canal bed deposits are finer than a No. 200 sieve.

TABLE 19.—MECHANICAL ANALYSES OF SUSPENDED SILT AT VARIOUS DEPTHS.

Item No.	Depth, in feet.	Percentage of silt by weight.	Percentage of Silt Passing and Retained on Sieves.					
			Passing No. 20, retained on No. 40.	Passing No. 40, retained on No. 60.	Passing No. 60, retained on No. 100.	Passing No. 100, retained on No. 200.	Passing No. 200, retained on No. 300.	Passing No. 300.
NEAR THE MIDDLE OF THE COLORADO RIVER AT YUMA, ARIZ., JULY 27, 1920.								
1	Top	0.310	0	0.26	0.97	4.84	10.13	83.80
2	4.0	0.456	0	1.05	5.94	15.72	17.51	59.78
3	7.0	0.479	0	0.64	4.40	14.37	20.19	60.40
4	10.8	0.614	0	1.09	9.12	16.82	21.22	51.75
IN THE MAIN CANAL AT HANLON, CALIF., 60 FT. FROM THE WEST BANK, JULY 26, 1920.								
5	Top	0.347	0	0.00	1.47	3.25	9.36	85.92
6	4.0	0.363	0	0.37	2.68	4.83	9.13	82.99
7	7.0	0.369	0	0.26	1.90	7.77	9.41	80.66
8	9.8	0.486	0	0.91	5.35	16.34	15.38	62.02

Perhaps the best indication of the weight of 1 cu. ft. of suspended silt (after drying), as carried by the Lower Colorado River, is shown by the data* collected by Mr. C. A. Engle, of the U. S. Indian Irrigation Service, at Parker, Ariz. Water is supplied to the Parker Project by pumping direct from the river into a settling basin. The weight of dry silt contained in 1 cu. ft. of wet deposit taken from the settling basin varied from 42 to 77.7 lb. and averaged 57.5 lb.

Several of the domestic water-works systems of Imperial Valley have settling basins which afford an opportunity to determine the weight and volume of "suspended silt" deposits.† Samples of these deposits taken by Mr. Peck show dry weights of silt per cubic foot of sediment ranging from 32.3 lb. for the softest material to 52.4 lb. for the most compact material, with an average of about 40 lb.

After a recent investigation of the quantity of suspended matter carried by the Colorado River, Mr. Howard stated‡ that suspended matter from daily samples collected for the U. S. Geological Survey, which were allowed to settle in tubes for a period of more than six months, showed a weight equivalent to about 34 lb. per cu. ft. of the deposited material. These samples did not contain the larger particles in their proper proportions, as the daily samples were taken from the bank. Samples of material from the bottom of

* *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 71.

† *Loc. cit.*, p. 70.

‡ *Water Supply Paper 636-B*, U. S. Geological Survey, "Suspended Matter in the Colorado River in 1925–1928", pp. 26–27.

the river collected in the experiments on the bed load showed a weight equivalent to 98 lb. per cu. ft. of the material after settling for four months.

Samples of river deposits* taken by Mr. McLaughlin and the writer along the Colorado River at Laguna Dam, Yuma, and Imperial Valley Heading, indicated that the unit weight of dry silt in 1 cu. ft. of river deposits ranged from 73.2 lb. for the finer material to 90.7 lb. for the coarser, with an average of 81.6 lb. The material was a mixture of bed silt and sediment formerly carried in suspension, taken from the river's edge where it had been deposited at a low stage. Samples of bed silt taken from canals of the Imperial Valley, about the same time, averaged approximately 100 lb. per cu. ft.

Determinations of the silt content of the river water are usually made on a weight basis. A "percentage of silt by weight" is equivalent to the grammes of dry silt contained in 100 grammes of water, and is derived by weighing the water and then the dry silt, and taking the proportion of the latter to the former. The engineer, from a practical standpoint, is more interested in the volume of silt than in its weight. In order to convert the results of silt samplings on a weight basis to volume, it is necessary to assume the weight of dry sediment in 1 cu. ft. of suspended silt.

After giving careful consideration to all available data, the writer and Mr. Fortier came to the conclusion that the average weight of dry silt contained in 1 cu. ft. of suspended silt, as carried by the Lower Colorado River, would be approximately 62½ lb.† The determination of this weight, while only approximate, simplifies the conversion of the silt content of water from a weight to a volume basis in that percentage of silt by weight equals the percentage by volume. The writer is of the opinion that this weight will approximate that of suspended silt deposited in the bottom of a reservoir created by a high dam, such as that at Boulder Canyon, provided the deposits are not given an opportunity to dry out and thus destroy their colloidal condition. However, a mixture of suspended and bed silt deposited on the delta of the Colorado River undoubtedly would dry out and compact sufficiently to approach a dry weight of 100 lb. per cu. ft. after a period of years.

Differences in opinion as to the weight of 1 cu. ft. of silt have resulted in estimates of the average volume of suspended silt that would be deposited annually in reservoirs on the Colorado River ranging from 80 000‡ to more than 250 000§ acre-ft. per year.

At the rate indicated by the higher estimate, the proposed Boulder Canyon Reservoir would probably be almost filled with silt in about 100 years, while at the smaller rate the deposited silt would occupy about one-third of the reservoir capacity in the same length of time, provided no other dams were built above it during this period.

A report‖ published by the U. S. Geological Survey indicates that most of the previous estimates are too low because the annual load of suspended

* *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 68.

† *Loc. cit.*, pp. 4 and 71.

‡ *Water Supply Paper 395*, U. S. Geological Survey, p. 222.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1480.

‖ *Water Supply Paper 636-B*, U. S. Geological Survey, "Suspended Matter in the Colorado River in 1925–1928", by C. S. Howard, p. 28.

matter computed from samples collected at Grand Canyon was considerably larger than the average annual load computed from samples collected at Yuma. This report states that the volume that will be occupied by the suspended matter is unknown, but that it seems probable that estimates based on 33 lb. and 86 lb. per cu. ft. of the deposited material represent fairly well the extreme conditions of deposition. On the basis of 33 lb. per cu. ft., the quantity of suspended matter carried past Grand Canyon in 1925-26 would occupy 313 000 acre-ft. if deposited in a reservoir; the volume for 1926-27 would be 617 000 acre-ft.; and the volume for 1927-28 would be 263 000 acre-ft. On the basis of 86 lb. per cu. ft., the corresponding figures would be 120 000, 237 000, and 101 000 acre-ft., respectively.

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NIAGARA POWER

Discussion*

By F. P. Williams, M. Am. Soc. C. E.

F. P. Williams,† M. Am. Soc. C. E. (by letter).‡—The outline of the power plants given in the paper not only shows the progress in the industry at Niagara Falls, but illustrates the tendency in other places to consolidate the generation of power at one location in one large plant. If space had permitted, Mr. Gibson would probably have described the difficulties, especially with bearings, in the early days which prevented development of the total head. There are many who will recall seeing the tail-water of some of the plants discharging half way down the cliff.

Limitation of Diversion of Water Under the Existing Treaty.—In mentioning§ the Chicago diversion as being one of the factors that led to the unequal diversion allotted under the treaty, Mr. Gibson doubtless gives weight to the discussions leading up to the ratification of the treaty. The treaty itself does not mention the Chicago situation. From evidence presented in the Courts, it appears that the large diversion made by the Sanitary District is illegal. Construction of works to take care of the sewage of Chicago and to relieve this condition is in progress in order to reduce the quantity to the necessities of navigation. The flow required for navigation purposes is being determined by the Supreme Court of the United States. In view of this it would appear that consideration of a greater diversion of water at Chicago than that required for navigation was without legal foundation.

Economic Benefits.—Mr. Gibson's view‖ that steam improvements have not advanced to a point that will enable all water-power plants to be discarded is well reinforced by the construction of such plants that is taking place at the present time in Canada, in some parts of this country, and abroad, especially in Switzerland. It is evident that there are some factors besides

* Discussion of the paper by Norman R. Gibson, M. Am. Soc. C. E., continued from December, 1929, *Proceedings*.

† Secy. Engr., New York State Water Power and Control Comm., Albany, N. Y.

‡ Received by the Secretary, November 13, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1735.

‖ *Loc. cit.*, p. 1739.

the comparison with steam-generating cost which determine the feasibility of hydro-electric projects. Stability of water power, which enables its cost to be computed accurately for a term of years, and freedom from fear of interruption which confronts steam-produced power on account of possible interference with fuel supply, are among such factors. In some cases these influences are of great importance.

Further development of power at Niagara is feasible, providing works are constructed in the river to distribute the flow; the development of water power on the St. Lawrence River is economic; additional regulation by storage reservoirs will result in more hydro-electric projects being developed on the interior streams.

The statement* that the company had not collected the difference between the cost of steam and hydro-electric power from the consumer, but a considerably less sum, calls attention to the industries that have been built up at Niagara Falls which require low cost power in order to exist. Until recently Niagara Falls was the center of the largest electro-chemical metallurgical industries on this Continent.

Ultimate Diversion.—Mr. Gibson has recognized the universal solicitude for the preservation of the scenic grandeur of the Falls. Credit for their preservation is due to the establishment of parks on both sides of the river by Government agencies; to the treaty for limiting the amount of diversion; and to the architects and engineers for the design of works so as to subordinate them and harmonize them with the surroundings as far as possible.

The author describes† the plan for increasing the diversion sufficiently for an economic development rather than attempting to determine the ultimate permissible diversion. The preservation of the grandeur of the Falls appears assured as long as there is as conservative a plan for increased diversion and as long as there is as frank a presentation of the various influences.

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1740.
† *Loc. cit.*, p. 1743.

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REGULATION OF LEVELS, FLOW, AND NAVIGATION, NIAGARA RIVER:

SUMMARY AND CONCLUSIONS OF VARIOUS STUDIES

Discussion*

By E. W. Lane, M. Am. Soc. C. E.

E. W. Lane,† M. Am. Soc. C. E. (by letter).‡—The control of the lake levels is a most interesting problem. The writer worked on the study of the regulation of the Great Lake levels made by the Joint Board of Engineers for the St. Lawrence Waterway, which was under the immediate direction of Colonel Pillsbury. He entered upon these studies with a predilection for the improvement of the existing condition by the construction of regulation works for the benefit of navigation and power development. This seemed better than the compensation of the lake levels for diversions by placing fixed obstructions in the connecting streams. Such fixed obstructions would benefit navigation by increasing the navigable depths, but would be of no advantage in the development of water power.

As a result of the studies, the writer is convinced that the Joint Board of Engineers got to the bottom of the matter, and shares with the author the view that the benefits at the present time do not justify the cost of regulation works. Since this conclusion was stated by the Board, the Supreme Court decision on the Chicago diversion, and the rapid rise of lake levels, have further strengthened the conclusion.

The studies made heretofore for the regulation of the Great Lakes have been with the view of improving the lake levels for navigation, or of causing an increased flow in the Niagara and St. Lawrence Rivers in low-water periods, or to accomplish both these results. Another benefit of regulation is possible, however, but so far as known, no studies have been made of the practicability of regulation for this end.

* Discussion of the paper by George B. Pillsbury, M. Am. Soc. C. E., continued from November, 1929, *Proceedings*.

† Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary, November 4, 1929.

Within transmission distance of the St. Lawrence and Niagara water powers are many existing and possible hydro-electric developments on streams which have a large flow at certain seasons of the year and low flow at others. The outflow of the Great Lakes varies in much the same manner as these streams, but it might be feasible to use the lakes, particularly Erie and Ontario, to store water during the season when the other streams had an excess, and discharge from storage when they were low. By the interchange of power an equalization of supply might be secured which would be of sufficient value to justify the expense of control works. In fact, it seems probable that some such result may come about as a natural sequence of events. When the International Section of the river is developed, it will probably be by means of a single dam at the lower end. For some years after this is completed there will be a surplus of power and a market for at least part of it will be found in supplying existing hydro-electric systems with power during their low-flow periods.

As the demand for power approaches the capacity of the International Section an effort will be made to continue to supply power during the low-flow season, to the existing hydro-electric systems, for since the value of secondary power is much less than that of primary power, the value per kilowatt-hour of power which will combine with the secondary power and provide a continuous supply, is considerably greater than the value of primary power. With this condition of the power demand there will be an immediate return on an investment in regulation works; and because of the immense quantities involved, they may be economically justified, especially since Lake Ontario will be already controlled by the power plant in the International Section and control works for Lake Erie are comparatively inexpensive. Because of the rapid changes taking place in the electrical power field, one would be rash to predict anything so far ahead as this condition must necessarily be; but to state that regulation would never be feasible would be equally rash.

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PULVERIZING OF SEWAGE SCREENINGS AT BALTIMORE, MARYLAND

Discussion*

BY MESSRS. WILLEM RUDOLFS AND H. K. PALMER

WILLEM RUDOLFS,† ESQ. (by letter).‡—A method for disintegrating comparatively coarse sewage screenings might be a valuable adjunct to large modern sewage disposal plants. The simple cutting device used in the Baltimore, Md., experiments described by Mr. Keefer§ seems to fulfill the requirements in a general way. An inspection of the machine in operation and observation of the material handled showed that the coarse materials were cut into small pieces which were easily carried by the sewage.

Mr. Keefer points out that there are two possible methods of disposing of the disintegrated screenings: (1) By dumping them again into the sewage and allowing them to settle in sedimentation tanks; and (2) by adding them directly to separate digestion tanks. He states‖ that "the latter method most likely will be adopted at Baltimore." There are advantages in both methods, but local conditions would decide which method is best suited. The method of sedimentation used in Baltimore provides for a storage of the fresh sludge for 8 to 12 days. The additional quantity of screenings would require an increased storage capacity. In plants where the sludge is removed daily or continuously the increased storage capacity would be negligible. The long time of storing fresh solids results in the production of very acid material. It has been found that acid production from screenings is even more intense and a mixture of fresh solids and screenings stored for 8 to 12 days would result in material still more difficult to handle and requiring more lime. With continuous sludge removal there is no chance for intensive acid production.

It is demonstrated that sewage screenings digest well when placed in tanks,¶ but the time required is longer than for fresh solids. If the screenings are taken out and discharged into digestion tanks it is possible that layers are formed in the tanks. Left in the sewage the distribution is more equal and no local zoning with possible disturbances would result.

* This discussion (of the paper by C. E. Keefer, M. Am. Soc. C. E., presented at the meeting of the Sanitary Engineering Division at Asheville, N. C., on April 21, 1927, and published in September, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf., Dept., Sewage Disposal, New Jersey Agricultural Experiment Station, New Brunswick, N. J.

‡ Received by the Secretary, November 14, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1760.

‖ *Loc. cit.*, p. 1765.

¶ *Proceedings*, Am. Soc. for Municipal Impvts., 1928, and *Sewage Works Journal*, 1929.

In calculating comparative costs,* Mr. Keefer takes a digestion capacity of 4 cu. ft. per capita. There is no doubt that under present Baltimore conditions 4 cu. ft. per capita or more is needed.

However, if the sludge is heated to 82° Fahr., this can be reduced to about 1 cu. ft. per capita. The amount of dry screenings produced at Baltimore is about 1 045 lb. per day. Assuming that the daily production of dry solids retained by a good settling tank is 50 grammes per capita per day, the addition of screenings to the flow would result in an increased digestion capacity of about 9 400 cu. ft. (at 82° Fahr.). Heating of the material, therefore, would result in a reduction of digestion capacity for screenings alone (at Baltimore) of from 34 720 to 9 400 cu. ft.

It may not be necessary to disintegrate fairly coarse screenings to prevent troubles with floating material, if the digestion capacity is large enough and the mass is kept at optimum temperature conditions. At plants where no heating facilities are available, or where the sludge compartments can not economically be heated (such as in Imhoff tanks), a shredding and cutting would undoubtedly be beneficial. The disintegration will take place in a tank, but more finely divided material decomposes more rapidly. In the case of very coarse screenings disintegration would be of decided benefit to the digestion and would help to prevent material from floating. It is for these several reasons that Mr. Keefer's paper is of interest to the profession; his experiments have called attention to a simple workable method that could be tried at other places.

H. K. Palmer,† M. Am. Soc. C. E. (by letter).‡—As the author states,§ this method seems to be a means of disposal that can be used profitably only under certain restricted circumstances. The quantity and quality of the screenings would be largely affected by the distance the sewage traveled. Where the distance is great most of the solids would be so comminuted that they would not be caught on a coarse screen, and those that were caught would be fibrous and more difficult to grind, thereby increasing the cost per cubic foot of screenings, although not per million gallons of sewage. If the sludge were to be used for fertilizer the addition of such material to the digestion tanks might be a disadvantage.

In a plant operated entirely by electricity the elimination of an incinerator is an advantage, especially where coal is used as fuel. If gas is collected from the digestion tanks it might be utilized, thereby disposing of a waste by-product and avoiding the expense of handling coal.

The grinding of screenings has been suggested elsewhere as a preliminary for ocean disposal, the ground screenings being returned to the screened sewage. Such a method is not to be recommended if there are any bathing beaches within a radius of several miles. Much of the material is such that it would continue to float indefinitely and always would be a source of pollution. In Southern California grease is deposited on beaches several miles from the sewer outfall, and ground fibrous material would be equally troublesome.

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1765.
† Chf. Draftsman, Los Angeles County Sanitation Dists., Los Angeles, Calif.
‡ Received by the Secretary, December 6, 1929.
§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1771.

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PAPERS AND DISCUSSIONS

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FILTERING MATERIALS FOR WATER AND SEWAGE WORKS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION

Discussion*

BY MESSRS. WILLEM RUDOLFS AND J. A. CHILDS

WILLEM RUDOLFS,† ESQ. (by letter).‡—In connection with material for sewage trickling filters it is stated in the Committee's report that experimental units are contemplated and opportunities to co-operate are appreciated. It is evident from the number of questions that require study, that no one experimental unit can be conducted so as to obtain results in the near future. The Committee cannot conduct experimental filters, nor dictate what type of experiments should be made by others, but there seems to be abundant room for co-operation in the way of suggestions and corroboration.

Some of the questions which require definite study are:

1.—Effect of Type of Material:
- (*a*) On durability.
- (*b*) Clogging.
- (*c*) Rate of application.

2.—Effect of Size of Material:
- (*a*) On purification.
- (*b*) Film formation and sloughing.

3.—Comparison of Most Effective Sizes of Different Materials, to Establish the Effect:
- (*a*) On rate of application.
- (*b*) Depth of filter.

In order to study some of these problems the New Jersey Sewage Experiment Station has built and put into operation a small experimental filter, at Plainfield, N. J. The filter is round, 12 ft. in diameter, and 6 ft. deep. There

* Dicussion of the Progress Report of the Committee of the Sanitary Engineering Division on Filtering Materials for Water and Sewage Works, continued from December, 1929, *Proceedings*.

† Chf., Dept., Sewage Disposal, New Jersey Agricultural Experiment Station, New Brunswick, N. J.

‡ Received by the Secretary, November 14, 1929.

are take-offs for sampling at the 1, 3, and 5-ft. levels and at the bottom. The filter is divided into four parts, which are filled with either crushed stone, slag, gravel, or limestone, all 1½ by 2½ in. in size. The analyses made include tests for bio-chemical oxygen demand, ammonia, nitrites, nitrates, suspended solids, ash, and bacteria, and observations are made on the growth of the stones. It is contemplated to run the filter in its present condition (definite size of stone and definite application) for at least one year. Changes in the rate of application are then to be made, followed by changes in the size of the filter materials. An intensive study of the working-in period has already produced some interesting results. It is perhaps unfortunate that four larger experimental filters could not be built, each receiving a full spray.

While the Committee cannot very well dictate the lines of attack for the different experimental units, a co-ordination of the work to be done at different places would be highly desirable.

Even if experimental units with the same general program were conducted at different places, it would be possible to emphasize at one place the more strictly engineering features, while at another the chemical and biological features could be studied in greater detail. The Committee could suggest which problems to emphazise and the questions which those interested would like to have answered first; it could watch the progress; and, finally, it could condense and annotate the available results. The writer has no doubt that those who are making the investigations will welcome any suggestions and help the Committee can give. When the results are brought together and commented upon by the Committee, it might not be possible to standardize practice. The Committee, however, should be in a position to point out the reasons for numerous difficulties and thereby prevent repetition of mistakes.

J. A. CHILDS,* M. AM. SOC. C. E. (by letter).†—The work of this Committee, so far as trickling filter material is concerned, has been directed toward the physical, chemical, and structural characteristics of materials commonly used, and the conclusions for the testing and selecting of materials for trickling filters appear to be basically sound. There is no conclusion as yet relative to the most suitable size of material for trickling filters. Possibly this omission has been for some good reason.

The relationship which any particular size, or range of size, of filter material bears to filter efficiency at various rates of application of sewage of different strengths may be of considerable importance in determining filter loadings. While reasons other than satisfactory analytical results may limit the quantity of sewage, or the loading which a filter may receive, nevertheless it appears that many filters are designed and operated at rates considerably below their actual capacity. This may be due to lack of complete and comprehensive information, which forces the designing engineer in self defense to select filter rates and loadings which he feels sure will produce satisfactory results. In other words, he may use a rather high safety factor. More complete and reliable information regarding all the factors which enter into filter operation

* Chf. Engr. and Secy., Metropolitan Drainage Comm., St. Paul, Minn.
† Received by the Secretary, November 29, 1929.

would enable the designing engineer to "hew to the line more closely", to the financial benefit of his client, not only in determining the size of any particular filter installation, but also in making comparisons between trickling filters and activated sludge plants as to their relative economy.

A study of the operating results of the trickling filters in Minnesota and elsewhere in the United States indicates quite a wide range of sewage loading. However, when the bio-chemical oxygen demand results of filter influents and effluents are plotted, it is found that the curve for any particular filter, generally speaking, follows a straight line. Practically all the data obtained tend to substantiate a general conclusion that the reduction in oxygen demand follows the general curve expressed as $R = K L$, in which, R is the reduction, L, the loading of the filter, both expressed in terms of pounds of 5-day bio-chemical oxygen demand per unit of filter surface, and K, is a constant within reasonable limits for the particular sewage and filter. This general conclusion appears to be applicable regardless of the rate of application up to, and in many instances probably beyond, the rates at which these particular filters were being operated.

It is probable that the value of K is influenced by many factors, among which are the characteristics of the particular sewage, the presence of industrial wastes, temperature, dosing cycles, nozzle size and spacing, depth of filter, types of under-drainage system, and the size and shape of the filter material itself. From the standpoint of the work of this particular Committee, it would be interesting to know what influence, if any, the size or character of the filter material has upon the value of K.

Designing engineers everywhere will appreciate the value of reliable data which will enable them to plan their treatment works so that each step will harmonize in its economical relationship with the other parts of the process. It is hoped that it will be possible in the Canton, Ohio, and other experiments to investigate more fully the relation of size of material, depths, and filter loadings.

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CHARGES AND METHOD OF MAKING CHARGES FOR PROFESSIONAL SERVICES

REPORT OF SPECIAL COMMITTEE

Discussion*

BY MESSRS. S. BENT RUSSELL AND WILSON T. BALLARD

S. BENT RUSSELL,† M. AM. SOC. C. E. (by letter).‡—The recommendations of this Committee appear to be conservative and quite sound. The writer would suggest some possible modifications with the idea of making them a more definite guide in fixing charges for service.

Engineers should strive to approach certain standards of charges so that clients would be assured of fair and impartial treatment and the engineers themselves get what is due them individually in practice and profit. To illustrate, each engineer might estimate a designer's time at $2.00 per hour whether he actually pays the designer on his staff more or less than that sum. A similar base charge could be made for a surveyor's time and another for a resident engineer having a given staff under him.

To make the principle of these suggestions more clear, the following additional recommendations are offered: That all engineers aim to approach certain standards in estimating the cost of serving clients; and that all standards of such costs be based on assumed ideal conditions.

Each assistant would then receive a fair salary based on the time required for adequate training for the service he renders and on his ability and merit. Under such conditions it may be assumed that each engineering office would be busy nine months in each year. All surcharge on assistants' pay and other overhead expense should be made, accordingly, under the "readiness to serve" idea.

The charge for personal service of the consultant and for profits should also take the lost time into account.

The personal service charge would then be in proportion to the consultant's standing in the profession and to his qualifications as an expert for the par-

* Discussion of the Report of the Special Committee on Charges and Method of Making Charges for Professional Services, continued from December, 1929, *Proceedings*.

† Cons. Civ. Engr., St. Louis, Mo.

‡ Received by the Secretary, November 7, 1929.

ticular service in the case and also, of course, to the importance of the case. It should not be scaled down in a price-cutting spirit on the one hand, nor raised to exploit a client on the other.

Under such conditions, the total profits for all consultants in a given territory should be sufficient to give them a fair living, up to the standard of other learned professions. Each consulting office should get a fair share of the total profits, but it should be in some proportion to the efficiency and merit of the organization. Each client should be charged a fair proportion of the total profits for the service he gets.

The writer suggests that the total charge recommended for service to each client be estimated as under the assumed conditions and be considered the "normal charge" for such service; and, furthermore, that all engineers be urged to use the "normal charge" as the guide in their practice.

Assuming that under some such plan, a schedule of normal charges might be approximated, the following rules should be made to be observed in principle by engineers for the general good:

> 1.—It is not ethical to attempt to secure employment by offering to do work for less than the normal charge in proportion to the estimated cost of the construction work because that is unfair to competing engineers.
>
> 2.—It is not ethical to take advantage of a client's special needs to ask more than the normal charge. Such unfair treatment of a client works injury to the reputation of the Engineering Profession.
>
> 3.—It is unethical for an engineer to offer his services, all or in part, free for the purpose of increasing his practice or advertising himself or his company, because such action might force competing engineers to do the same in self-defense.

Salesmanship is another item that seems pertinent since it has a bearing on the subject of charges. It has been given much emphasis lately. Of course, an engineer may employ the salesman's art to a certain point with propriety. An excessive use of the art, however, tends to lower the dignity of the profession and also probably to lower its efficiency. In view of this the following rule is suggested to be added to the three previously mentioned:

> 4.—It is unethical for an engineer to endeavor through superior salesmanship to exploit the public in any way or to outwit his competitors so as to secure more than his due share of practice or employment. Such action is unfair and hurtful to competing engineers.

One more point: On consideration it will be seen that if engineers are to co-operate in any way to regulate charges, great care should be taken to guard against inviting the suspicion that they are trying to exploit the public through greed. It may be noted that trades unions have sometimes overdone the business of raising wages when they had the power.

It is devoutly to be wished that a large proportion of Society members will take an active interest in this discussion so that the final report on the matter will yield a true consensus of opinion. The writer hopes that the suggestions offered herein may prove of some service in arriving at conclusions of value.

WILSON T. BALLARD,* M. AM. SOC. C. E. (by letter).†—It is the firm conviction of the writer that men and owners generally are willing to pay a fair and adequate price for services or commodities the value of which they can understand, appreciate, and interpret in terms familiar to themselves. This discussion, therefore, will be limited to three points, namely, the manner in which engineers should accept employment; unfair competition; and education of engineers and clients.

In the building construction field, particularly, structural designers are frequently employed by contractors or architects at a meager rate of pay. Engineers should not accept employment in this mannner. Every bridge, building, or other structure, large ar small, should first be safe, built to endure, and designed so as to permit of economic construction as well as to render convenient and economic service to its owner. It is the function of the engineer to plan it so. Second, a structure should possess beauty in keeping with its purpose and surroundings, and, in the case of a building, should have its space and volume disposed so as to provide maximum convenient utility to its owner. To plan it in such a manner is the function of the architect. Finally, it should be well constructed in accordance with the plans and specifications. This is the function of the contractor and should be performed under the supervision of engineer and architect. The owner, therefore, should employ separately the engineer, the architect, and the contractor, and they, with him, and among themselves, should co-operate in the accomplishment of his purpose with credit to themselves and their separate callings; and each should be recognized for the results for which he is responsible.

Unfair competition of college professors and civil employees with practicing engineers is a condition that, in time, should produce its own cure. The deans and trustees of engineering universities and colleges, and public officials, on the one hand, and employers of practicing engineers, on the other, will, no doubt, ultimately realize that no man can satisfactorily or efficiently fulfill two important and essentially different occupations at one time, for he must surely neglect one in favor of the other. A college professor has neither the time, equipment, nor proper organization, and seldom the necessary experience in practical matters, to handle, promptly and efficiently, engineering work other than his teaching and research. It should be stated here that with the facilities at his hand he can, and, in a great many instances, does render the profession eminently valuable contributions through research.

One argument, possessing some merit, advanced in favor of professors and instructors, undertaking outside work is to the effect that it fits them to instruct their students properly. If this be true the aspirant for a chair in an engineering faculty should, upon his graduation, spend several years in the employ of practicing engineers and contractors whose work lies in the field in which he has chosen to teach and thereafter return whole-heartedly to the work of teaching and training his students. He can readily keep himself abreast of engineering progress by well-chosen reading of the technical press, and interpret what he reads to his students in the light of his past practical

* Vice-Pres., the J. E. Greiner Co., Baltimore, Md.

† Received by the Secretary, November 11, 1929.

experience. A practicing engineer who is handling work of importance must be ready to go when and where required, and to give of his time without limit in serving his clients. This is impossible for the professor or civil employee.

The education of engineers and especially clients will require much time, patience, and tact. Engineers must be taught to respect their profession and to practice it on a high and honorable plane; otherwise they cannot hope to command the respect to which it is entitled. It is an unpleasant admission, but nevertheless true, that at present there are those who do not seem to hesitate to violate the fundamentals of the Code of Ethics. These violations should be reported, promptly investigated, and the guilty parties, if members of the Society, should be disciplined by it.

Engineers should avoid indulging in public criticism of the work of fellow engineers, unless the principles of engineering have been so flagrantly violated as to endanger life, health, or property. Even then it were better to convey the criticism to those with authority and power to correct, because such criticism must frequently be in terms unintelligible to the layman, and, therefore, serves only to destroy confidence in the profession. Constructive criticism and discussions within the Society are welcomed by all right-minded engineers and should be freely indulged in to the benefit of all.

As the Committee report points out,* the education of engineers in proper professional conduct can best be accomplished while they are yet in school and college, and before they meet the rough and sometimes disillusioning elements of competition. In school, the idealism of youth is at its highest tide and impressions received then by those whose characters are of such quality as to fit them honorably to pursue the profession of engineering will be crystallized into strong, permanent convictions.

The education of clients can be accomplished best by keeping before them the clean, honorable, intelligent conduct of members of the Engineering Profession. There is no way in which clients can be brought together in groups and instructed. They must learn to distinguish between the results and value of good, competent engineering service as compared with mediocre or careless work, and the value of good work must be impressed upon them in their own terms, usually dollars. When this lesson is thoroughly learned by them, they will be quick to realize that the additional 1% or 2% in the fee of the competent engineer is insignificant when compared with the savings of many kinds affected by his work. It must be admitted that this is a lesson learned more readily by clients of the other professions, for example, the law, for upon the successful conclusion of a case a lawyer's client can see at once and can frequently bank the results, whereas the true value of sound engineering work often requires years to make itself evident.

Clients must be continuously impressed with the fact that the proper practice of engineering is done on the rigid and honorable principles of a well-conceived code of ethics. This can be better accomplished by precept than by preaching. Having learned it, clients will soon come to realize that competent and valuable service is to be expected only from those engineers who hold their profession and its code of ethics in high regard.

* *Proceedings*, Am. Soc. C. E., September, 1929, Society Affairs, p. 283.

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EXPERIMENTAL RESEARCH ON VIBRATION DAMPERS AND INSULATORS

Discussion*

BY MESSRS. J. P. DEN HARTOG AND FRANCIS G. TATNALL

J. P. DEN HARTOG,† ESQ. (by letter).‡—The paper deals with a subject of great interest and on which little research has been done. The general problem of vibrations in structures as well as in machinery is quite complicated, as Mr. Slocum remarks, and it is too often discussed in a purely empirical manner. It is possible, however, to give a perfectly rational explanation of many of these phenomena, including the one which is the subject of this paper.

The author states§ that "no practical solution has yet been found of the vibration characteristics of such a primary element as a simple beam of uniform cross-section under load." This is not entirely true, since Rayleigh's method‖ gives, with very little calculation, a solution which is at least 95% correct; it is a "practical solution." If one wants to put some more work into the problem, Ritz's method‖ can be used, giving a solution in a series form to any desired accuracy. Both these methods are extensively used by vibration engineers.

The experimental set-up of Fig. 1¶ can also be completely calculated if the full data concerning the masses, flexibilities, and damping constants of the various members are known.

These items are not given in the paper. The elements that are least known in this set-up are the "load from the testing machine" and the flexibility of the supports between the I-beams and the platform.

* This discussion (of the paper by Stephen E. Slocum, M. Am. Soc. C. E., published in October, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought to the attention of all members for further discussion.

† Research Dept., Westinghouse Elec. & Mfg. Co., East Pittsburgh, Pa.

‡ Received by the Secretary, November 6, 1929.

§ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2109.

‖ A description of Rayleigh's as well as of Ritz's method can be found in "Vibration Problems in Engineering," by S. Timoshenko, D. Van Nostrand Co.

¶ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2111.

The "load from the testing machine" consists partly of mass or dead weight and partly of spring pressure. These two causes have the same effect in compressing the test pad; but they behave radically different, dynamically. From the results in Fig. 2 (*a*),* however, certain tentative conclusions can be drawn.

In the first place, the curve in Fig. 4† with the data in the description of Fig. 2 (*a*) makes it possible to calculate the spring constant of the test pad, which is 3 000 lb. per in. of deflection. This is so flexible as compared to the H-beam, that the latter flexibility can be safely neglected. The system of the test pad, H-beam, and testing-machine head then can be represented by Fig. 15.

Let k_1 (= 3 000 lb. per in.) represent the spring of the test pad, while M and k_2 are the equivalent mass and spring caused by the H-beam and the testing-machine head. If this system is subjected to a vibratory motion, $a \sin \omega t$, at its bottom end, the mass, M, has a motion, $b \sin \omega t$, in which,

$$b = a \times \frac{k_1}{k_1 + k_2} \cdot \frac{{\omega_n}^2}{{\omega_n}^2 - \omega^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Equation (3) is the principal formula of the spring vibration absorber. In this formula, ${\omega_n}^2 = \frac{(k_1 + k_2)}{M}$, is the natural frequency of the system.‡

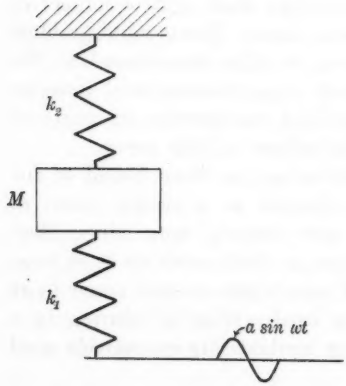

FIG. 15.—SCHEMATIC ARRANGEMENT OF TEST PAD, H-BEAM, AND TESTING MACHINE HEAD.

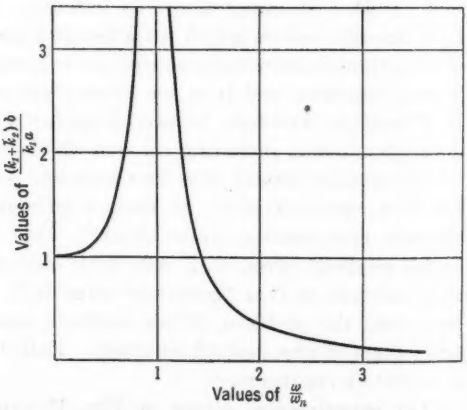

FIG. 16.—CURVE SHOWING THE RATIO, $\frac{b}{a}$, OF THE AMPLITUDE OF THE RECEIVER TO THAT OF THE TRANSMITTER AS A FUNCTION OF THE FREQUENCY OF THE INPUT VIBRATION.

The ratio, $\frac{b}{a}$, of the amplitudes of the "receiver" and the "transmitter", as calculated from Equation (3), has been plotted in Fig. 16. The curve shows that for very slow motions ($\omega = 0$) the ratio, $\frac{b}{a}$, is equal to $\frac{k_1}{(k_1 + k_2)}$; that is, the "static" condition. At a certain speed, ($\omega = \omega_n$), resonance occurs and the

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2114.

† *Loc. cit.*, p. 2115.

‡ The complete theory can be found in the *Electric Journal*, Vol. XXI (1924), p. 160, in an article by C. R. Soderberg and, also, in the book by Timoshenko previously noted, p. 33.

motion becomes infinite theoretically. For very fast motions of the bottom end ($\omega = \infty$), the mass, M, is too inert to follow it, and its amplitude is zero.

It is seen that the vibration absorber works only when its impressed frequency is high with respect to its natural frequency, which is another way of stating that a soft spring is needed. It is seen that for a stiff spring the absorber may become injurious; that is, the received vibration may be larger than the transmitted vibration. The test results indicate that they were conducted at the extreme right of the diagram of Fig. 16. In fact, it is possible that in the present test set-up there is some low running speed in which the amplitudes received are larger than those transmitted.

In case the unbalanced motor had been placed on the H-beam directly above the test pad, while replacing the equivalent dead weight of the motor in its original location on the I-beams, the vibration of the I-beams would still be larger than that of the H-beam (at speeds from 1 100 to 1 500 rev. per min.). In this case, again, the result would be a "negative damping".

If, instead of corkboard, steel springs of the same flexibility had been used, the results would have been very much the same, since the tests were taken for points that do not coincide with the critical speed of the system in Fig. 15, so that internal friction of the material is of no importance.

The resonant frequency of the I-beams cannot be appreciably affected by the remainder of the system since the flexibility of the test pad is so great. A rough calculation of this frequency from the available data indicates a higher frequency than the records show. A possible explanation for this is that the supports between the I-beams and the test platform have some flexibility, or that the test platform itself is not entirely steady.

When the corkboard test sample is replaced by sand or gravel, which have a higher modulus of elasticity, the k_1 of Equation (3) becomes larger and, consequently, the condition is displaced toward the left in Fig. 16, thereby showing a larger transmitted vibration.

Summarizing, it can be stated that proper vibration absorption can be accomplished fundamentally by supporting the vibrating system on such springs that the natural frequency of the system becomes low with respect to the frequency of vibrations. This can be done by steel springs or by organic materials like cork, rubber, etc.

The writer has had some experience with these absorbers as applied to machinery, and mostly has used steel springs, for the reason that present knowledge concerning organic materials is insufficient. It is quite possible to mount a structure on corkboard and get good immediate results, but it is not known what will happen after five years. The material may become stiffer with time and thus become worse than nothing. (See Fig. 16.)

A fundamental research of the vibration-absorbing qualities of cork, rubber, felt, sand, etc., should consider the following points:

(1) Flexibility as a function of the number of repetitions of the vibration, of the load, and the time applied.

(2) Internal hysteresis of the material under the same changing factors as before.

(3) Model tests of the type described in the paper, but with springs and masses which are definitely known, so that the results can be checked by calculation.

This program is quite involved and will take considerable time and effort, but it is believed that the data thus obtained will be of great help to the profession.

FRANCIS G. TATNALL,* ESQ. (by letter).†—The subject of vibration is of importance as a new field of engineering and, by this time, should have its specialists and a quantity of experimental knowledge. This does not seem to be the case, however, because little definite information on vibration is available and few specialists seem to be in the field. This field, it seems, is a fertile place for a committee of the Society.

Vibration studies are a specialist's job, because even though frequency and amplitude of vibration can be fairly well catalogued for any structure, the more difficult question is encountered: "What is to be done about it?"

Damping and insulating are not the answers. These appear to be only makeshifts, usable until the time comes when vibration can be measured and predicted so that it can be cared for by proper design. The completed structure, or parts of it that can vibrate as a whole, must be designed so that their natural frequency will conveniently avoid any forced vibration that will tune in to set up resonance.

With existing apparatus the frequency and amplitude of vibration can be measured under actual working conditions. With this knowledge, the machine or structure under consideration can be re-designed, and then must be tested again to see whether its natural frequency is high enough or low enough to keep out of step with external forced vibration. There is too much "cut-and-try" in this method so that ninety-nine out of a hundred times it is still a laboratory procedure. In some few instances, this method is actually applied to commercial design.

The paper dealt with low frequency, large amplitude vibration. The method is a good laboratory test for comparing vibration dampers. Very much higher frequencies, taken from structures in the field under actual operating conditions, can be measured with existing apparatus. Furthermore, vibration can be recorded from a large number of points simultaneously. The double resistor type of electric telemeter has been frequently used for such purpose and to good effect.‡ This instrument is attached to the member to be investigated and vibration in the member changes the resistances of a pair of highly refined carbon resistors which proportionately unbalances a Wheatstone bridge. An oscillograph, connected across the apex of the bridge, records faithfully the frequency and amplitude of vibration. If twelve telemeters, for instance, are attached by very small cap screws to twelve points on a structure and their lead wires brought together at a panel board and connected to the twelve elements of a single oscillograph recorder, twelve simultaneous vibration

* Mgr., Test Equipment Div., Southwark Foundry & Machine Co., Philadelphia, Pa.

† Received by the Secretary, November 11, 1929.

‡ See "Recent Developments and Applications of the Electric Telemeter," by O. S. Peters, *Technologic Paper No. 247*, U. S. Bureau of Standards.

records can be made on a single 12-in. film, and the frequency can be obtained by comparison with a thirteenth record. This record may be that of 60-cycle alternating current, a break-circuit Swiss chronometer operating a reflecting kicker, or a special tuning-fork marking off 0.01-sec. intervals. The equipment is first calibrated so that known values of amplitude have been established. A magnification of four hundred times can be obtained with this method, and the amplitude of vibration is measured from the actual wave on the film. The principle of measuring electrical unbalance on so sensitive a circuit as a Wheatstone bridge eliminates all inertia effects except that of the oscillograph vibrator which is infinitely small.

Frequencies up to 1 200 cycles per sec. can be recorded at the sensitivities requisite for this work, although frequencies as high as 8 000 cycles per sec. have been recorded. The frequency is limited by the natural period of the oscillograph vibrator. As sensitivity in recording increases, frequency decreases. In other words, the greater the magnification of amplitude, the less the frequencies that can be recorded. Most vibrations encountered in commercial engineering structures are less than 300 cycles per sec., so this permits high magnification. Errors in recording range from 3 to 10%, depending on the severity of vibration.

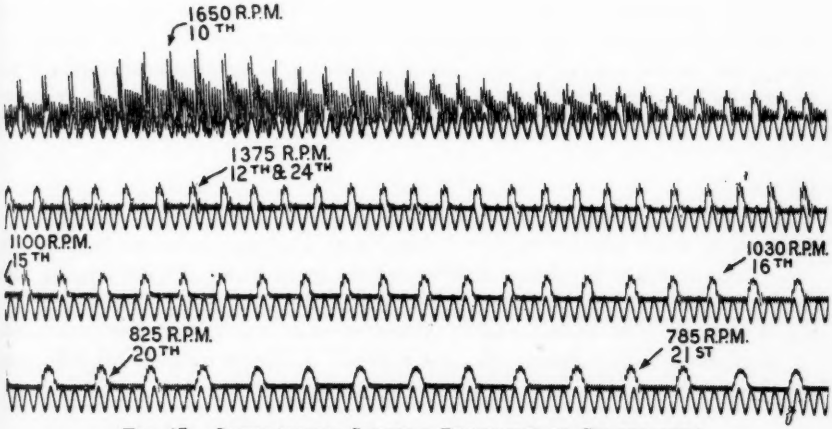

FIG. 17.—OSCILLOGRAPH SHOWING RESONANCE OR SYNCHRONISM.

Perhaps the telemeter can furnish additional information on the subject of vibration. Perhaps it can extend the work of Mr. Slocum, so as to record the actual damping effect, in structures under service conditions, of the materials he has studied in the laboratory. It can surely pick out the resonance or synchronisms as illustrated in Fig. 17.* When resonance is attained, the oscillograph trace can be seen to rush to it as a peak and becomes fainter (see position marked 10th harmonic 1 650 rev. per min., top line, Fig. 17). The heavy regular wave is the timing trace. The superimposed wave is the forced vibration in the part. When the natural frequency of the part is not

* As recorded by the Cleveland Wire Spring Co.

approached this wave is regular and well defined, but when synchronism is reached, it flies all to pieces. The figures 10th, 12th, 15th, etc., designate the harmonics and the speeds noted are the rates of forced vibration.

By means of the telemeter, vibration can be measured, recorded, studied, and catalogued, but it must be remembered that there the function of the telemeter stops. What remedies to use to overcome the phenomenon is another problem entirely.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SPILLWAY DISCHARGE CAPACITY OF WILSON DAM

Discussion*

By Cecil E. Pearce, Assoc. M. Am. Soc. C. E.

Cecil E. Pearce,† Assoc. M. Am. Soc. C. E. (by letter).‡—As the author states,§ there are relatively few data in regard to flows through large spillways as deep as 18 ft. When spillways are designed for such heads, one might, in calculating the discharge of a large rounded ogee spillway section, use a conservative value, say, $c = 3.4$, and hope that the spillway would discharge at least as much as computed by using that coefficient in the formula:

$$Q = c\, L\, H^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Therefore, when such measurements as those presented by the author appear, engineers who are interested immediately greet them "with open arms" and analyze them to see if there are data which may be of use to them.

Equation (7)‖ is in a form which makes the arithmetical work laborious, and a simpler equation would be desirable. The equation of a parabola would have afforded such an equation. In this case, the equation of a parabola is:

$$x^2 = 4\,p\,(y - A) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Similarly, to the author's solution, when $x = e$ and $y = B$,

$$(4\,p) = \frac{e^2}{B - A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and, finally, after integrating:

$$Q = A\,L - \left(\frac{D}{12}\right) L^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

* This discussion (of the paper by Louis G. Puls, Assoc. M. Am. Soc. C. E., published in October, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Designing Engr., Water Dept., Pasadena, Calif.

‡ Received by the Secretary, November 11, 1929.

§ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2131.

‖ *Loc. cit.*, p. 2137.

in which,

$$D = \frac{A - B}{e^2}$$

which equation is much easier of arithmetical solution.

A computation shows that Equation (7) (the ellipse equation) gives the same values as Equation (12) (the parabola equation) in the fifth significant figure.

The writer does not agree with the author in his mathematical treatment of "Discharge Capacity of Spillway with Adjacent Gates Open."* Mr. Puls has stated that "this increase in quantity discharged, can be treated as if it were due to an increment of head."

It seems more logical to the writer to attribute the increase in discharge due to adjacent gates being open as caused by a lessening of the end contractions, because it can easily be seen that with only one gate open, the water would flow diagonally into the spillway opening, as well as straight into it. This would tend to cause more end contraction than when the adjacent gates are open and the water is flowing more nearly straight through the several spillway openings. If that is correct reasoning, the relation between Q (one gate open) and Q' (adjacent gates open) depends upon formulas having to do with end contractions. Regarding end contractions, Hoskins† states:

"Francis concluded that each contraction decreases the effective length of the weir by an amount proportional to H, and gave the following formula:

$$Q = 3.33\ (b - 0.1\ n\ H)\ H^{\frac{3}{2}}$$

in which (n) is the number of end contractions, usually two."

Other authors such as King‡ and Gibson§ also cover the same subject, stating in effect that nothing better has yet been found, and that the formula seems to be rational.

Quotations from these and other authorities will not be made herein, but are mentioned for the purpose of showing that, from the best information available on the subject, the discharge does not change in proportion to the three-halves power on any value of $\left(\frac{H}{H + \Delta h}\right)$, but end contractions diminish the discharge from Q (without end contractions) to an amount equal to $\left(\frac{L - n\ k\ H}{L}\right) Q$.

It is thus seen that the change in discharge is not even proportional to H, but for any given values of H and L, it is a constant with L in the denominator, such that if L is large enough, the change occasioned by end contractions is small in proportion to the total. For these reasons, the writer believes that Equation (8)‖ does not correctly show the relation which exists between Q and Q'.

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* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2139.
† "Text Book on Hydraulics", L. M. Hoskins, Third Edition, 1911, p. 149.
‡ "Handbook of Hydraulics", Horace W. King, First Edition, p. 67.
§ "Hydraulics and Its Applications", A. H. Gibson, Second Edition, pp. 141-142.
‖ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2140.

The author also states* that "the discharge, Q', can be found because all other values are now known." Even if Equation (8) were correct, the absence of any mention of any measured value of h_3 leads one to believe that the author has assumed $h_3 = h_1$ and calculated Q' accordingly from Equation (8), obtaining 10 848 sec-ft.† The writer believes that it is not necessarily correct to assume $h_3 = h_1$ in the case of gate fully open, and, therefore, h_3 and Q' are both unknown; so that for any correct solution based on experiment, the value of h_3 should also have been measured. However, even if the value of h_3 had been measured, it is believed that Equation (8) is irrational and could not be expected to give rational results.

The derivation of Equation (8) implies that Q' is to be calculated from it. Otherwise, it has no use. Therefore, it is assumed that Q' was calculated and not obtained from actual measurement of velocities over the whole section, and that, therefore, the value, $Q' = 10\,848$, cannot be used as a check on Equation (8).

If the author's presentation is correctly understood, the writer believes that insufficient data are presented for any reasonably accurate calculation of Q' (gate fully open) which could purport to depend on the field observations; or, at least, if sufficient data do exist, the mathematical presentation given by the author does not solve the problem.

The author's determination of $c = 3.420$ for one gate open, however, is as nearly correct as the data warrant, and for conditions similar to those at the time of measurement at Wilson Dam, the value of c found ought to be fairly reliable.

The author, by the use of his "vertical-velocity" and "vertical-velocity-area" curves, and his method of attack in integrating the area under the ellipse, has shown considerable ingenuity, and has presented the data very clearly and simply. He also deserves commendation for making the data which he has collected available for others.

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* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2140.

† *Loc. cit.*, p. 2141.

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ESSENTIAL FACTS CONCERNING THE FAILURE OF THE ST. FRANCIS DAM

REPORT OF COMMITTEE OF BOARD OF DIRECTION

Discussion*

BY MESSRS. EDWARD GODFREY, C. A. P. TURNER, AND LARS R. JORGENSEN.

EDWARD GODFREY,† M. AM. SOC. C. E. (by letter).‡—No man's opinion as to the cause of the failure of the St. Francis Dam is of any value. This is for the reason that it is not a question for opinion. Where facts are so clear that they leave no escape from definite conclusions, there is no room for opinion. It is the writer's intention to show that this failure is in that category.

One may ask, what is the volume of a rectangular block of stone, having certain known dimensions? No opinion is of any value in answer to this question. If the answer does not agree with the product of the three dimensions, it is an opinion, and is valueless. If it does agree with that product, it is not an opinion, and it is correct. An opinion may rightly be expressed as to the weight of that same block of stone, if its specific gravity can only be judged.

A bridge may fail, say, in a tension member of a certain known sectional area and tensile strength, due to a load that produces a certain calculable stress which equals or exceeds the known strength of that member. The cause of that failure is not a matter for opinion; it is a mere question of fact. Engineers should recognize the difference between subjects for opinion and subjects for conclusions based on facts. Where possible, reports on failures should set forth the facts so clearly that conclusions are inescapable.

The St. Francis Dam was in reality three dams joined in one. Two of these were of identical type; the third was essentially different. Neither of the six reports pointed to this fact. It is the outstanding feature and any report that fails to emphasize it, fails completely to show the lesson taught.

* Discussion of the Report of the Committee of the Board of Direction on Essential Facts Concerning the Failure of the St. Francis Dam, continued from December, 1929, *Proceedings*.

† Structural Engr., Pittsburgh, Pa.

‡ Received by the Secretary, October 21, 1929.

The two wings of the dam had a base width 65% of the height; the middle portion had a base width of about 80% of the height, as one can see by laying a straight edge down the stepped slope on Fig. 2.* The middle part, furthermore, was underdrained. Personally, the writer does not place much store in under-drainage of dams. He does, however, recognize the enormous significance of 80% of base width.

The greater relative base width unquestionably places the middle part of this dam in a class apart from the two wings. It is of a different type of design. Separated from the other two dams that made up the St. Francis structure, every engineer would be compelled to admit that its stability is greatly superior to the others.

A very simple calculation will show that, assuming water to weigh 62.5 lb. per cu. ft. and concrete 150 lb. per cu. ft., a triangular dam, to be stable against up-stream water pressure alone, will have a base width about 65% of the height. Hence, a dam of this cross-section is of the type designed for up-stream pressure only.

An equally simple calculation will show that if the triangular dam has a base width of 85% of the height, it will be completely stable against overturning due to full up-stream pressure and full under-pressure on the entire base. A dam of this cross-section is of the type designed for all the water pressure that can be exerted against it. Any dam approaching this cross-section is of a type designed for part of the under-pressure.

The 65% base is a distinct line. There is no factor of safety. If there is any under-pressure whatever, the up-stream edge of the base is opened for the entrance of water; this is an entering wedge for the admission of water and a greater upward pressure, and the beginning of failure. This is not opinion. It is demonstrable fact, based on accepted laws of mechanics.

Surely, then, a design that takes care even of some under-pressure is in a class apart from one that would fail with the first appearance of such pressure. At the expense of apparent repetition the writer has emphasized the difference between the middle dam and the two wings. The middle dam stood up. It had to be blasted away. It stood the enormous shock of the two wing-dams being broken off and floating away and the dynamic force of the tremendous flow of water when the wing-dams moved off. It is quite possible that the 0.7 ft. of movement measured in this middle section occurred while the two wings were floating and resting on water alone, the only frictional resistance being that offered by the middle section.

There is nothing in the six reports on this failure and the facts stated concerning the dam to indicate that the rock under the middle of the dam is in any manner whatever different from that under one of the wings. Yet these reports go so far as to state that the standing part of the dam is proof of the correctness of the accepted method of design, which is the method used for this dam (which includes the wings).

There is no recognition whatever in these reports of the tremendously significant fact that the middle, standing part had a relatively wider base than the two wings that floated away and that this is the sole difference.

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2150.

Engineers, especially those who deal with dams in their design and who write books on the subject, fail, in a large measure, to appreciate the floating effect of water pressure. The idea seems to persist that there can be no flotation or under-pressure unless there is tail-water to produce it, that a thing cannot float unless it is entirely surrounded by water.

Before the failure of the St. Francis Dam, water was leaking rapidly from beneath the dam. The leakage was so great that the gauge showed a measurable drop in water level. It is significant to note that it was a general leakage and not a "blow-out" or a succession of them. The leakage was watched. No man would "watch" a blow-out; he would run for his life.

The fact that there was leakage proves conclusively that there was pressure —pressure under the base of the dam. It is a simple fact of mechanics that water could not work through 100 ft. or more of rock or dirt without pressure behind it; and it is also a simple fact of mechanics that water exerts pressure equally in all directions, not merely in the direction in which it is traveling. This means, as a demonstrated fact, that there was a pressure exerted in an upward direction under the base of this dam, amounting to hundreds of millions of pounds; and it was absolutely unprovided for in the two wings of the dam that failed.

There is no room for opinion here. The report of the Committee of the Board of Direction states that only provision for up-stream pressure was made in the design of this dam. Because a designer ignores a gigantic force in designing a structure, it does not follow that that force ceases to exist for that structure.

There has been much experimental evidence proving that water does exert pressure under the base of a dam, particularly in two recent papers published by the Society.* Cut-off walls, sheet-piling, grouting of foundation, under-drainage—all of which were supposed to inhibit under-pressure—have been resorted to in dams subjected to actual tests,* but still the pressure was there.

The specious argument that full hydrostatic pressure could not exist against the base of the dam, because the several grains of sand or dirt must have some measure of contact where water would be excluded, has been proved false by actual tests in cylinders.* The proponents of this argument failed to see that there might be several layers of dirt particles and that one would push the other, thus making up for the suppositional lack of contact. The two papers referred to were published before the reports on the St. Francis Dam were made. They ought to quiet forever every advocate of the total or partial neglect of under-pressure.

The argument that water penetrates soil by capillarity and flows out without pressure is quite contrary to the known laws on this subject. The St. Francis Dam floated out.

The Committee reports give the weight of concrete as 140 lb. per cu. ft. With a base 65% of the height, the resultant of up-stream pressure and under-pressure intersects the down-stream edge of the base, as indicated in Fig. 6.

* "Upward Pressures Under Dams: Experiments by the United States Bureau of Reclamation," by Julian Hinds, M. Am. Soc. C. E.; and "Hydrostatic Uplift in Pervious Soils," by H. de B. Parsons, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1527 and 1317, respectively.

The full weight of the dam, therefore, was supported and carried by water. It was actually floating. Mere water current under such circumstances would not account for the movement of 10 000 tons of rock in one piece ½ mile. Flotation would fully account for this movement, which would have been physically impossible if the base of the dam had approached 85% of the height instead of being exactly the width for complete flotation.

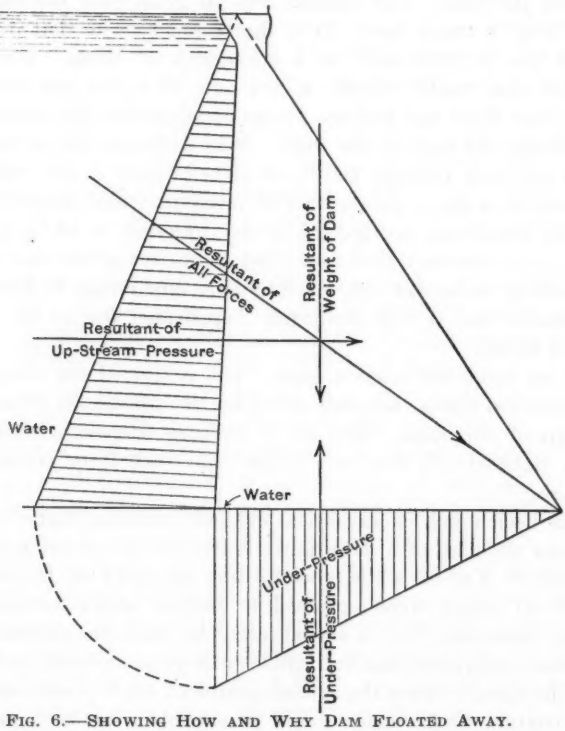

FIG. 6.—SHOWING HOW AND WHY DAM FLOATED AWAY.

The floating of wood paving block is doubtless responsible for the many failures in that type of paving. A bucket of water would suffice to float a large area of blocks.

Modern engineers need to learn from the ancient Egyptians the facts of flotation. There is scarcely any doubt that they floated the great stones of the Pyramids to place on barges, either through excavated canals or canals built up by temporary retaining walls, or both.

One of the reports points out that the concrete of the dam was much stronger than the rock on which it rested.* The inference is that a rock of less strength than concrete is inadequate for a foundation. There is no reason why the rock should be any stronger than necessary to perform the service required, which is to bear the weight of the dam. The maximum load on 1 sq. ft. of the soil at the base of this dam is 31 700 lb., which is the weight

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2153.

of 220 cu. ft. of concrete (220 lb. per sq. in.). The foundation rock, according to tests made by one of the Committees, had a compressive strength of 523 lb. per sq. in. Here is a factor of safety of more than two, and, in fact, any material in place and confined has much greater compressive strength than a free cylinder. There is no possibility that this weight could crush the rock of the foundation.

One Committee report* attributes the failure to a blow-out in this rock. Conceding that the rock was soft, as rocks go, and that it was particularly soft when it was wet, what about the tremendous pressure that would be required to force out a mass of this rock, for instance, from a hydraulic cylinder? Where did it come from? What about the concomitant cylinder pressure? Could a hydraulic cylinder force out a great mass of rock and not be subject to wall pressure? If this sub-foundation had a wall pressure, what about the effect of this pressure on the overlying dam that is conceded to be totally lacking in stability against an ounce of such pressure? What held the dam down when this high-pressure work was going on? It would be illuminating if some one who sets up this "blow-out" theory would explain how it acts.

The writer concedes that a blow-out is a possibility in soil. When soil can be rendered liquid or semi-liquid by water penetration, and when such fluid soil is covered with a compacted crust or pavement, the fluid pressure will accumulate—having the head of the water source—and will lift the pavement or crust, allowing the fluid mud to escape. A failure by undermining might happen to a dam of ample width built on mud or quicksand. It could not happen if the soil approaches the nature of rock. The Galveston Causeway failed in this manner. The sea dashed up in great waves and saturated the high ground. This water, tending to flow back to the sea, accumulated a head under the paved slope, lifted the pavement and undermined the Causeway.

Earth dams fail sometimes by blow-outs. Paved or compacted downstream slopes are subjected to accumulated under-pressure; and when the "crust" is lifted by this pressure, there is a subterranean flow of mud that may even cause the crest of the dam to sink. The impression is then given that the weight of the crest is too great for the underlying soil to bear.

Reports on failures of earth dams have attributed the failure to the weight of the earth in the crest, when the disturbance was initiated in the shape of a blow-out many hundreds of feet from the crest. One such case was the Lafayette Dam in California.† In this dam intense subterranean water or mud pressure was proved to exist below the base.

There is serious need of revision and re-alignment in engineering literature and thought on the subject of water pressure and flow of semi-liquid mud; and the occasion of a serious disaster such as the St. Francis Dam failure is an appropriate one to point out such need and the way it should trend.

The middle part of the dam was moved 0.7 ft. when the wings floated away.‡ Instead of being an indication of instability this is a remarkable proof of stability. When this part could withstand the enormous shock of the

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, pp. 2154 and 2159.
† *Engineering News-Record*, January 31, 1929, p. 190.
‡ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2151.

two wings breaking away and the dynamic force of the moving body of water, and suffer merely a movement of a fraction of a foot, it is the most conclusive demonstration ever staged of the complete stability of a dam properly designed for under-pressure.

If the whole dam had been of a base width of 85% of its height, it would be standing to-day in spite of all that the reports say regarding the rock formation. No more convincing proof of this could be conceived than the action of the middle section which only approached that proportion, but which well exceeded the danger line of 65 per cent. Nature cannot put enough water behind and beneath a triangular dam of 85% base width to move it away.

One of the reports states* that separation of the foundation on inclined cleavage planes was responsible for the failure. These cleavage planes were transverse with the dam. How could there be sliding in this direction? It is true that after the dam was washed away and great holes were eroded under where it had stood, there were landslides in a transverse direction, but this could not happen when the dam and its underlying rock were in place.

It is of little consequence which wing of the dam gave way first. They were both equally weak and both were floated away by the known and proved pressures of water.

There was a time, some years ago, when under-pressure seemed to be becoming recognized as a fact and something to be provided for in the stability of a dam, but some utterances of recent years seem to indicate a reversion to the attitude of earlier years, when under-pressure was totally ignored. A new book on structural engineering† treats under-pressure as something of little consequence and something that can be readily avoided.

One argument for ignoring under-pressure was based on the fact that an engineer many years ago computed that a dam to be designed for under-pressure and ice pressure should have a base width of 352 ft. for a height of 250 ft.‡ Another argument is the statement that certain dams are said by their designers to have been designed with no regard to under-pressure. One of these is the New Croton Dam. The cross-section of this dam is sufficient for full under-pressure in any horizontal joint. The designer's idea is not the thing that determines the stability of a dam; it is the concrete placed in it.

Again, the opinion has been expressed§ that,

"* * * whether or not uplift should be considered, is a question for the exercise of good judgment on the part of the engineer. * * * Where large populations live below a dam and where great loss of life would occur in case of failure, the engineer could be justified in providing a much larger factor of safety than would be required in ordinary cases."

It would have been an enormous saving on the part of Los Angeles, if there had been an inflexible law requiring that this dam have the 30% additional mass concrete that spells the difference between precarious existence and absolute stability permanence.

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 2153.
† "Structural Engineering," by George Fillmore Swain, 1927.
‡ *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 256.
§ *Loc. cit.*, p. 260.

Another engineer also adds* the weight of his opinion in this tendency to ignore under-pressure. He states that "he has no direct personal knowledge of any failure of an upper portion of a masonry dam by over-turning or sliding on a lower portion except where caused by ice thrust." The Gleno Dam in Italy sliced off at a lime mortar joint above the foundation, and 500 or 600 people perished. He states,† furthermore, that with "a barrage of drain holes one may reasonably expect the intensity of uplift pressure to approximate that due to tail-water." The tests previously mentioned refute this theory.

C. A. P. TURNER,‡ M. AM. SOC. C. E. (by letter).§—The St. Francis Dam was built according to old and established traditions of the Civil Engineering Profession in an ultra-expensive manner such that its ponderous mass would have rendered it safe had the underlying formation been impervious hard rock. The occurrence of impervious hard rock formation is unfortunately the exception rather than the rule in suitable dam sites. Inspection of the foundation bed too often stops with (1) superficial investigation of the apparent hardness of the material; (2) the difficulties of cutting it out in the "dry" instead of subjecting it to the erosive action of a small hydraulic jet under a pressure of 200 lb. per sq. in.; and (3) forming the final decision and verdict from the results of such exposure. Such an investigation in the construction of the St. Francis Dam might have led to making a deeper cut in the formation or, perhaps, to the application of the bridge builder's method of injecting neat-cement grout into the underlying strata under pressure of 150 lb. per sq. in., thus sealing the pores and crevices in the underlying material at relatively small expense. To the extent that such filling is effectively done a dam of the dimensions of the St. Francis structure might have been safe had the underlying rock been capable of carrying the load without shearing or crushing; but should the judgment as to the impregnation be as faulty as that of the sufficiency of the underlying material then the superstructure, otherwise safe, could disintegrate with disastrous results.

While weakness in the natural formation under both the eastern and western parts of the dam was sufficient to have caused failure of the structure, the great loss of life and damage to property resulted from the characteristic deportment of the plain concrete forming the superstructure. When partly undermined, it cracked into fragments that toppled one over the other, thus permitting a huge wall of water, perhaps 100 ft. high, to rush down the valley, spreading death and destruction in its path.

During the last decade those engaged in building and bridge construction have learned, by experience, the superior toughness and dependability of reinforced concrete over plain concrete. They find that, with one-fifth the mass, greater strength can be secured, with a dependability largely independent of cracking or checking of the concrete when resisting flexure by combined bending and twisting.

* *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 275.

† *Loc. cit.*, p. 276.

‡ Cons. Engr., Minneapolis, Minn.

§ Received by the Secretary, October 23, 1929.

Some years ago the writer designed a warehouse having a roof slab over the elevator shaft, 22 ft. square and nearly 7 in. thick, reinforced with small rods in both directions. The contractors of the sprinkler system built a steel frame carrying a 50-ton water tank about 18 ft. above the slab. The tower of ample strength for vertical load, plus a 50-lb. horizontal wind pressure, was deficient under an upwardly inclined wind force of less than 20 lb. It collapsed and dropped 50 tons of water and tank on the concrete roof slab. The slab was cracked through, bulged down 6 in. under the point of impact, and two of the small rods each way were stretched to the breaking point. Nevertheless, the cracked slab carried its excess load, and the water slopped over the side of the building, demolished a section of the canopy over the loading platform, and knocked the ends out of a couple of box cars on the adjoining track.

For safety, a concrete dam should have the same toughness as this roof slab. If undermined, cracked, and buckled out of shape, the cracked segments should hold together; they should carry the hydraulic pressure and prevent, by remaining intact, the sudden emtying of the dam, and devastation and loss of life, such as occurred in the case of the St. Francis structure.

The slab in question proved its capacity to absorb a thousand times its elastic work of deformation in kinetic energy of impact; and yet, although slightly disfigured and cracked through under the point of impact, was still capable of carrying three times its design load without further damage or injury. This resistance arises from the fact that with multiple-way reinforcement twisting resistance in dished-slab areas is reduced only a small amount by cracking of the concrete from bottom to top in multiple directions. As long as the broken fragments held in place by the steel are squeezed together in the compression zone of the plate so that rotation about the vertical cannot occur, the twisting resistance to flexure remains operative with nearly the full force and effect that subsists before cracking by sharpness of curvature under flexure takes place.

Because building code authors, textbook writers, and university doctors are, in general, unfamiliar with the mechanics of torsional strength of plates in flexure this kind of resistance has been erroneously credited to the tensile strength of concrete after it has broken in tension.*

To secure the safety and resilience of the roof slab in dam construction the slab should be inclined at an angle of 40° to the horizontal so that the hydrostatic pressure tends to hold the dam down instead of tipping it over as in the case of vertical up-stream faces of the gravity type. The slab should be supported by inclined columns distributing the weight of concrete and water over a greater area of the soft rock bottom so that the bearing pressure is reduced a safe amount, not more than one-third that involved in a massive gravity structure.

The average shearing resistance of trap granite, limestone, and sandstone is about one-fifth the compressive resistance of the rock, and with laminated mica schist the ratio is probably not more than one-third as favorable. Accordingly, artificial faulting of the soft underlying rock was very likely caused

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1560.

by the excessive concrete weight of the St. Francis structure permitting the infiltration of water at the base of the dam. Softening and crushing of the schist under over-load would naturally follow and would be accompanied by the flow of saturated and powdered material comparable to the action of quicksand under pressure.

The apparent safety of the massive gravity type of concrete dam proved misleading in the case of the great dam at Austin, Tex., which was founded upon an apparently hard limestone formation and was carefully designed. Fairness to the eminent designers of these and like structures should cause others to question whether in like position following traditional practice they would have afforded any surer protection to the public. If there be a reasonable doubt, its existence constitutes an indictment of present practice and accentuates the necessity for rational safeguard in improved methods of design and construction.

The fact that safety against devastation by flood through faults in the natural formation under high dams is attainable at half the construction cost of the massive imitation of safety embodied in many gravity structures should add a commercial incentive to the demand for better and safer work.

Compare, for example, the 6-ft. thickness of a reinforced concrete plate ample for a 200-ft. head at the base, with the 170-ft. thickness of the dam that failed; or, again, compare a liberal thickness at the top, of 3 ft., with the 16-ft. thickness of the collapsed structure. Against this favorable comparison there is to be debited the one and one-half times greater inclined height of the slab, the high grade of concrete required, and the additional cost of reinforcement, columns, and footings.

In conclusion, the documents from which this report is condensed are open to the constructive criticism that their substance is that of a pusillanimous complaint that Nature did not make the site suitable and adaptable to a gravity type structure. They should have pointed out the obvious fact that the ingenuity of the designer directed along conventional lines, did not produce a structure adapted to that particular location. Protection of property from the floods common in that district required a safe dam at or near that location which would catch and impound a sudden heavy fall of rain. The problem of protecting the community is to treat a foundation of that character and to build so that the structure can operate safely even though there may have been some oversight in properly cutting off the ground-water beneath the structure. To do this efficiently, as the writer views it, requires only the simple expedient of injecting into the underlying material neat-cement grout at a pressure one and one-half to two times that of the hydraulic pressure which will be brought upon the underlying material later when the dam is filled. Under such pressure, with the drill holes spaced not too far apart, this cementing material will find its way into every pore and crevice that could be impregnated by pure water. The operation is simple and easily performed. It is not so difficult that it requires scholarship and mathematical attainment to achieve. What the designer in dam construction should aim at is true safety under all conditions. A semblance of safety in mass concrete and the impressive appearance of its bulk is too often a delusion and a menace. This is conclusively

shown by the unfortunate failures of the dams at Austin, Tex., St. Franciscito, Calif., Johnstown, Pa., and many others, demonstrating the substantiality of the criticism.

LARS R. JORGENSON,* M. AM. SOC. C. E. (by letter).†—In practically all the various reports stress is laid upon the existence of a poor foundation under the St. Francis Dam, and this fact is given as the dominating cause of the failure.

The disaster undoubtedly proved that a dam having dimensions like the upper part of the St. Francis Dam should not have been built on this kind of foundation. It was practically also proved that a dam of more ample cross-section was safe since the middle and heavier part was still standing after the sides, having the slim sections, went out.

The Commission appointed by the Governor states‡ that "there is nothing in the failure of the St. Francis Dam to indicate that the accepted theory of gravity dam design is in error." If this statement can be construed to mean that a dam having a base width of 66½% of the height—like the sides of the St. Francis Dam—comes under the accepted theory of gravity dams, the writer feels that statement should not go unchallenged.

The design of a gravity dam is not such a simple matter as it may seem to be. The name, "gravity dam," is often being applied to any structure having a more or less triangular cross-section, where the base width varies all the way from 66½% to nearly 100% of the height.

With a weight of concrete of 150 lb. per cu. ft., and a cross-section having a base width of 66½%, such a dam only under the most ideal conditions would be a gravity dam, with a factor of safety of a few per cent. more than 1 if built straight in plan. If arched in plan, the factor of safety can be expected to be increased in the summer and decreased during the late winter months.

Some gravity dams have been built recently with a base width 66½% of the height. Fortunately, they have been built on good rock. As gravity dams, they hardly have a factor of safety of more than 1 and how much more they actually possess depends upon how much tension the concrete in the up-stream face will stand. The weakest point very likely will be at the junction between the rock and the concrete.

Under conditions, ideal as to uplift, pressure, etc., a gravity dam that is triangular in section with a base width 66½% of the height, may have a factor of safety slightly more than 1 when built in a U-shaped canyon or on a site with flat side slopes. If such a dam is built in a V-shaped canyon, with steep sides, however, the factor of safety will not be 1 uniformly along the profile. It will be less for the part of the dam higher up on the slope.

The writer is not aware that this important fact has ever been considered seriously in a gravity dam design. It is, however, very simple to understand that it must be so. If the deflection of the dam in a down-stream direction, due to the water load, is investigated at various places on the side slopes and in the middle where the dam is highest, it becomes apparent.

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* Cons. Engr., Constant Angle Arch Dam Co., San Francisco, Calif.

† Received by the Secretary, November 18, 1929.

‡ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2155.

Using the deflection formula proposed* by Fred A. Noetzli, M. Am. Soc. C. E., for a cantilever with load increasing uniformly from zero at the top to a maximum at the bottom and applying this on a section 1 ft. wide with parallel sides: $Y_{max.} = \frac{P \times H^5}{E \times T_0^3}$, for a 100-ft. section = 0.0049 ft.; and for a 205-ft. section, $Y_{max.} = 0.0128$ ft. The deflection of the crest for the high part of the dam is 2.6 times greater than that for the 100-ft. cantilever on the hillside, approximately 100 ft. away horizontally. It is certain that the vertical shear and the horizontal beam action between the two points of various deflections will tend to iron out this variation by transferring the load from the high sections in the middle to the lower sections on the hillside.

As the St. Francis Dam had a base width of 66½ ft. where it was 100 ft. high, it only had a factor of safety of a little more than 1 at this elevation under the most ideal conditions, and when additional load was transferred from the middle to the sides, as the water rose, the sides failed because the total load introduced tension in the up-stream face, which could not be taken care of by the foundation. The foundation had considerable compressive strength, since it had supported the dam with reservoir empty, but evidently no tensile strength; undoubtedly uplift developed and helped to slide or turn over the two wings.

The fact that the sides were loaded with more than the water load directly upon them was sufficient reason for their failure since the factor of safety of the section was theoreticallly only slightly more than 1 without this additional loading. Had the rock been good there is no reason to doubt that the dam would still be in place, not as a plain gravity dam or as an arch dam, but as a cantilever in bending, and inasmuch as the factor of safety of such a structure is uncertain, a gravity dam with a base width of 66½% of the height should not be used when it is presumably to stand by its weight.

The St. Francis Dam was curved in plan and the contours of the sides lend themselves very well to this plan in so far that the abutments were practically radial in direction; but unless such a dam is provided with vertical contraction joints which are grouted later, at the proper time, the dam is made weaker by arching during the period of maximum contraction in the late winter.

Grouting the vertical contraction joints is not a cure for all ills and the maximum beneficial effect from it can only be procured by grouting at the proper time which is not immediately after the completion of the dam. Unfortunately, much grouting of vertical contraction joints is being done soon after completion because this is either the most convenient time or because the structure is a diversion dam in which the load will never be taken off after it is once applied.

The St. Francis Dam failed on March 12, 1928, and it is believed that this is about the time the dam body would possess a minimum temperature and, therefore, would be weaker than a straight dam of the same cross-section. The section of the dam in the middle had a base width of about 82% of the height

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 7, Equation (4*a*).

and if the sides had had the same width most likely the dam would still be standing. For dams of moderate heights a heavier section is needed much more on the steep hillside than in the middle and to this fact the civil engineer does not seem to give all the consideration it deserves.

It costs more to build on a poor foundation than on a good one, since a poor foundation requires a much heavier section to make a safe structure. The resultant of all forces should not cut the base just inside the middle third in any case, and if a dam is to be supported by a poor foundation the resultant of all forces should cut the base much closer to the middle.

There is no equivalent substitute for weight in a gravity dam. Cut-off walls and drainage help to make the dam safer, but if the section is cut much below that of a triangle having a base width 75% of the height it is not a gravity section under all conditions possible, having a factor of safety, positively, of more than 1, acting as a gravity dam. More than that is not necessary, but on a poor foundation it will require a base width of more nearly 90% of the height before assurance can be had that the dam will stand under any condition. The St. Francis Dam failure seems to prove this since the middle section stood the severe punishment to which it must undoubtedly have been subjected.

It becomes necessary at times to build dams on poor foundations and just because a fault runs through a dam site is not sufficient cause for abandoning a project if the fault is dead. Many rivers were made by digging themselves into fault lines, because that was the softest place they could find, and many dams are built across such rivers.

Especially on poor foundations, the excavation ought to be of sufficient depth to leave a shoulder in a down-stream direction. This will bring the shearing strength of the concrete into play, which is a much better factor on which to depend than friction.

On good foundations, stepping, so as to make the down-stream end of the steps higher than the up-stream end, will also put shearing into play as the principal factor preventing a movement of the dam in a down-stream direction.

The St. Francis Dam may have failed due to a number of causes, but the fact that the part of the structure located on the side hill had a factor of safety of less than 1, acting as a gravity dam, is sufficient reason alone for the failure of this part.

There has been quite a dispute as to which side failed first. As the two sides were equally weak there is very little doubt but that they failed at the same time and when the rising water level had introduced tension in the up-stream face.

The St. Francis Dam disaster should teach the Engineering Profession that a base width of 66½% of the height is not sufficient for a dam depending for its stability upon gravity action, but that a width of more nearly 75% should be considered as minimum for safe design on good foundations.

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PAPERS AND DISCUSSIONS

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ADJUSTMENT OF TRANSIT AND STADIA TRAVERSES

Discussion*

By Lowell O. Stewart, Assoc. M. Am. Soc. C. E.

Lowell O. Stewart,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer agrees with Mr. Rappleye that it is desirable to place the corrections for transit and stadia traverses in the lengths only; but he would add a few words in the way of caution lest one overlook some of the field conditions that may have an important effect on the results.

In the first place the example chosen seems to be at the very upper limit of accuracy in error of closure. Probably such accuracy would not be obtainable by ordinary methods with many of the transits in common use. As a rule the probable variation either way from the chosen reading is much more than 1 ft. at a distance of 1 000 ft.

Moreover, this variation is a sort of systematic error which seems to depend upon the individual. It has been found in many kinds of instrument work that an observer tends to "under-read" or to "over-read", on each observation. He rarely alternates. This seems to be true of stadia readings. The discrepancy will vary, approximately, with the distance because, as the length of sight increases, the intervals between the larger divisions would bear the same apparent relationships to one another as did the more minute divisions at the shorter distances. It would be wise, then, for an engineer who proposes to make an important stadia traverse to test his readings at various measured distances.

It is assumed, of course, that the stadia interval is known accurately. Many transits are functioning with home-made cross-hairs and the interval may or may not be the conventional "100", which is commonly used.

Atmospheric conditions have an important effect on stadia readings. On long sights the line of sight must pass near the ground where the refraction is

* This discussion (of the paper by Howard S. Rappleye, Assoc. M. Am. Soc. C. E., published in November, 1929, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof., Civ. Eng., Iowa State Coll., Ames, Iowa.

‡ Received by the Secretary, November 20, 1929.

uncertain and the readings correspondingly unreliable. In the same way a line over a plowed field on a hot day would be affected differently from a line over a pasture, or over a body of water.

All the foregoing factors have an important bearing on any adjustment. In fact, their effect should be reckoned and applied before an adjustment is attempted. Unhappily, some of these factors are not always known with accuracy. In the latter cases one should be cautious about placing too much faith in a rigorous method of adjustment, such as the author has set forth in a thorough manner.

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FACTORS GOVERNING THE LOCATION OF AIRPORTS

Discussion*

By Messrs. Stephen Child and Morris Knowles.

Stephen Child,† M. Am. Soc. C. E.—Tucson is a thriving city of about 40 000 population, in Southern Arizona, approximately seventy miles from the Mexican line. It has been actively and definitely "air-minded" for some time, as the following facts furnished the speaker by the Secretary of the Tucson Chamber of Commerce, will abundantly testify.

The first municipal aviation field in Arizona, if not in the United States, was established at Tucson, and began to operate as such in 1917. It had an area of 80 acres and cost $7 000. It is about 2 miles directly south of the city on a site selected by Eric Nelson, one of the world-famous fliers. About 1922, the Chamber of Commerce, in co-operation with the City, erected a hangar and a building for quarters for the employees who were furnished by the U. S. Army Air Service, and a lease was made by the City to the Air Service for the operation of the field.

This original airport of Tucson is still owned by the City and is leased to a commercial aviation company, which is offering to the public aviation training as well as taxi service. This provision meets the requirements of Mr. Baker, who recommends‡ that such features should not be permitted at the more official airport of the city.

In 1925, however, the Chamber of Commerce took much more important action and was able to secure the passage of a bill by Congress, setting aside two full townships (1 280 acres) only a little more than 4 miles from the center of the city. This land has been leased to the City of Tucson for twenty years, with an option to renew the lease for another twenty years, for the exclusive purpose of developing aviation. It will be seen that the important provision of permanency stressed by the author, is assured.

* This discussion (of the paper by Donald M. Baker, M. Am. Soc. C. E., presented at the meeting of the City Planning Division, at San Diego, Calif., on October 4, 1928, and published in November, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2307.

The tract is very nearly level, having a gentle downward slope from east to west of only about 1 per cent. There is no obstructing vegetation, no ledge or boulders to be removed, or important gullies to be filled up. There is merely the customary desert growth of mesquite and cactus. The topographical conditions, therefore, are nearly perfect.

Of this large area 240 acres have now been quite fully developed by the City of Tucson and the Army Air Service, at a total cost of about $30 000—a minimum figure. A hangar has been erected, attendants' quarters have been constructed, a pumping plant for water supply has been installed, and a Government Radio Meteorological Station has been set up and fully equipped. By means of a public subscription, aided by the City Government, sufficient funds to install a beacon light and flood lighting equipment have been raised. The Army Air Service keeps a sergeant and one private stationed at the port to service all Army planes. A large gas storage equipment has been installed by the Government, and private oil companies have set up storage tanks on the field for commercial planes. The equipment is, therefore, practically perfect for a port of this size, and there is ample opportunity for its extension. The airport is used jointly by Government and commercial planes, under the direction of the Army Air Service and the supervision of the City itself. Legislative control as emphasized* by Mr. Baker, therefore, is well provided.

As to accessibility it will be noted that the Tucson Airport is only about five miles from the heart of the city over existing rectangularly arranged roads, which are well paved. It should be emphasized, however, that the main line of the Southern Pacific Railroad crosses this airport at its southwest corner; that there is now a flag station stop at this point, which can be readily made more important when the need arises. There is no doubt that there will be constructed in the near future a diagonal avenue parallel to the existing railroad tracks, shortening the distance from town by at least a mile.

One of the well-known aviation transportation companies operates a passenger service between this port and Los Angeles, Calif., three times a week, a stop being made at Phoenix, Ariz., en route. Connections have been established so that it is possible now to leave Tucson at 8:00 A. M., arriving at Los Angeles at about 1:00 P. M., and connecting with the airplane service operating between Los Angeles and San Francisco so as to arrive there in the early evening.

Leases for hangar space are arranged with the City and the airport is now so well known and completely equipped that it has a double "A" rating, and is considered one of the best inland airports of the country. Between January 1 and October 1, 1928, the registration of the airport shows more than 900 airplanes using its facilities, and during the month of September, 1928, 275 were registered. The City of Tucson has appropriated for the fiscal year 1928-29, funds for the construction of a building to be used as a waiting-room, ticket office, and lunch room.

The author has suggested* a means of comparing different sites, and the speaker has thought it interesting to develop the rules given, as shown in Table 1.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2314.
† *Loc. cit.*, p. 2315.

TABLE 1.—Comparative Rating of Airport at Tucson, Ariz.

Factor.	Weight.	Percentage.	Rating.
(*a*) Topography	10	100	1 000
(*b*) Physical features	14	100	1 400
(*c*) Meteorology *	14	95	1 330
(*d*) Location with respect to community served	10	95	950
(*e*) Transportation facilities	11	95	1 045
(*f*) Accessibility	11	95	1 045
(*g*) Legislative control	5	100	500
(*h*) Utilities available at site	5	100	500
(*i*) Cost	10	90	900
(*j*) General desirability from all standpoints	10	95	950
Total	100	....	9 620

* No fogs; sun shines on an average of 358 days in the year.

In other words, the field is 96.2% ideal, which is a very high rating. It will be interesting to determine the ratings of other airports by this means of comparison. The speaker hopes that Mr. Baker will submit such comparisons, but it seems clear that for a comparatively small community of less than 40 000, the facts indicate a very creditable state of affairs for Tucson as to aviation and airports.

Morris Knowles,* M. Am. Soc. C. E.—It is well to emphasize the fact that airport location, like other new municipal problems, is simply one of city planning, and as such it should be solved. It is inconceivable to consider an airport as wisely located, if in the selection of a site the fundamental principles of good community planning have been violated. This has been stressed by Mr. Baker.† A definition has been given‡ that:

"An airport is a locality, either water or land, adapted for the landing or taking off of aircraft, and which provides facilities for shelter, supplies, and repair of aircraft; or a place used regularly for the receiving or discharging of passengers or cargo by air."

Safety and convenience, in the provisions for the landing or taking off of planes, are matters of first importance. However, aside from possible emergency fields, which may be for this purpose only, an airport assumes an objective for traveling planes other than that of landing; or to reverse the situation, a plane in the ordinary course of events must have some object in being at a certain location other than the mere fact of desiring to leave there. In short, this reduces to the statement, by way of emphasis, that while at present air transportation may be considered more or less in its infancy, its real importance as a public carrier certainly will increase and, with it, the necessity of locating the airport with respect to traffic-handling facilities and other similar features.

Mr. Baker has stressed ten features† requiring consideration in determining ratings for locations offered as sites for airports. In reference to five of these, he states that little consideration should be given to a site which does not measure up to 80 or 90% of the standards given for them in the tables.

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2309.

‡ *Bulletin No. 40*, U. S. Dept. of Commerce.

Although he does not say so, it may be assumed that he considers others of less importance. This suggests the possible elimination of such minor items as Factors (*g*)* and (*j*)* in the rating and the substitution of others of more importance. For example, there are two such items discussed under the heading of "Topography" (Factor (*a*)):† Those of open spaces for emergency landing fields and landmarks for identification.

The selection of a site for the Pittsburgh (Pa.) Airport, by a Committee of the Chamber of Commerce, made use of a rating schedule, developed after approximately two hundred sites had been listed for consideration. Many of these had been suggested by the public through a questionnaire which was sent out for the purpose. This, in order to confine suggestions within reasonable limits, was made up so as to attract attention to the salient factors which would be considered in the choice.

The study of this large list of possibilities was simplified by the preparation of charts which enumerated, on a comparative basis, the desirable and undesirable features of each site. This naturally eliminated many because of some outstanding objection or obstacle. The rating schedule was then developed and applied to the more desirable sites after they had been inspected from the ground and air. Thus, as the study progressed, eliminations increased, until only a few sites remained for consideration.

Such a rating table suggested as an aid in the final analysis, gives the weights of the various factors considered as important under the particular circumstances. It is as follows:

	Weight.
(1) Size, physical features, and freedom from obstructions..	20
(2) Distance from business district and City Post-Office....	15
(3) Accessibility by various transportation methods.....	14
(4) Possible emergency fields existing near-by.....	12
(5) Railroad connections, with sites for shops.....	10
(6) Freedom from wind, fog, and smoke.....	9
(7) Property and cost of converting (quite approximate)...	8
(8) Seaplane facilities in proximity.....	5
(9) Distinguishing landmarks for visiting pilots.....	4
(10) Water supply and fire-fighting facilities.....	3
Total	100

While such ratings form an important basis for selection, many general factors will still remain to be evaluated, and the final recommendation should depend on the experience and judgment of those who are attempting to make the selection.

In discussing the development of airports it is important to realize that the problems differ in coast and inland cities, not only because of topography, but because of location. For example, Mr. Baker emphasizes‡ the desirability of locating the field on the side of the city from which the aircraft will approach. This is not important for most inland locations, as the approach may be from any direction and, probably, in the majority of instances, unless detours are made, it will be necessary to fly over the city either approaching

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2309.
† *Loc. cit.*, p. 2310.
‡ *Loc. cit.*, p. 2313.

or leaving the field. At the same time it is possible that where the routes are at right angles—east and west and north and south—a location adjacent to the city at one of the intermediate points of the compass would make it possible to avoid flying over the built-up territory.

Mr. Baker refers* to the undue weight which often is given to the necessity of locating an airport close to the business center. This undoubtedly is true. However, not only is the element of time important, but the cost of additional transportation as well. Studies in some places have emphasized the fact that the cost of the additional hauling for mail, express, and passengers, over a distance of 10 miles as compared to 2 miles, may approach the amount of interest on the additional money which would be needed to purchase and develop a field near the center of the city. Commercial aviation perhaps has not yet reached the point, in this country at least, which would economically justify the clearing of a built-up area in the city for an airport. Nevertheless, such conditions may be anticipated with the perfection of helicopters, just as it has been found necessary in the past in the rebuilding or relocation of railroad terminals.

The City of Pittsburgh has some unusual problems in the development of an airport—because of topography and the difficulties of making a field relatively level—but when these have been solved it will have some peculiar advantages because the field will be high in the air and will overlook most of the surrounding country. This elevation is important in regard to the question of fog and smoke; it also makes it possible to provide sufficient graded land, adjacent to the field, so that the parking of automobiles and the erection of shops and convenience buildings can take place without infringement on the clearance adjacent to the field. In other words, because of the resultant lower elevation for such purposes, there is no necessity for removing structures, or for parking cars, seven times their height horizontally from the field; or, as an alternative, decreasing the effective area of the available space by this amount.

Referring to the requirements of ample land area for future growth, it is notable that no mention is made of that for the parking of automobiles. This is an important necessity and there should be ample provision on four sides of the field, so that at times of great public demonstration cars may enter and leave the parking area without interference with other traffic. It is well to emphasize this need of provision for ample area, at least for later expansion, if required. The probabilities are in favor of additional land being needed for industries associated with air transportation, and for similar purposes. Aside from every other consideration, there is one convinciong thought which gives assurance as to the probability of providing too large an area in view of anticipated improvements in airplane design. In the history of cities in the United States there have been few, if any, instances where too large an area has been set aside for public use.

One more item to which it seems desirable to call attention is that of the provisions for seaplanes. While this is an important matter for sea-coast

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2313.

towns, and those along the Great Lakes, it does not give the impression of being important for inland cities, even though they may have a good river harbor. The infrequency of such travel (due to the fact that most inland cities must utilize other forms of airships and that the interference of bridges is likely to be troublesome) does not suggest that much weight need be given to this element.

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HIGH DAMS

A SYMPOSIUM

Discussion*

BY F. G. SWITZER, ASSOC. M. AM. SOC. C. E.

F. G. SWITZER,† ASSOC. M. AM. SOC. C. E. (by letter).‡—In reading the interesting paper by Mr. Wiley,§ three points were noted with which the writer cannot agree. These may be summed up as follows:

First.—If a gravity dam is designed as a straight dam with the resultant line of pressure at the third point, then the same section built as an arched gravity dam will show no tension.

Second.—Arch action is always present in an arched gravity dam.

Third.—There can be no vertical stress in the concrete as a result of water, under pressure, in the voids.

The writer believes that the first and third of these points are contrary to fact and that in the second point the word, "always", should be replaced by the words, "not often."

As a matter of experience, the writer recalls an existing gravity dam, for which the first designs were those for a straight dam. The contours in the river bottom made it desirable to arch the dam in plan in order to utilize the best abutment sites and, at the same time, avoid a deep hole in the river bed. With the same section, computations showed that in the section reduced by convergence of the radii, tension would be present for some appreciable distance from the up-stream face. This design assumed full uplift pressure on the whole area at all elevations and with a maximum intensity equal to the full hydrostatic head. The section, therefore, was increased, and no reliance was placed upon possible arch action.

* This discussion (of the Symposium on High Dams presented at the Joint Meeting of the Construction and Irrigation Divisions, San Diego, Calif., October 4, 1928, and published in November, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further consideration.

† Cons. Engr. and Prof. of Hydr. Eng. School of Mech. Eng., Coll. of Eng., Cornell Univ., Ithaca, N. Y.

‡ Received by the Secretary, November 21, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2318.

In order to examine somewhat more closely into the problem, the writer has considered the simple triangular section of a gravity dam, as shown in Fig. 6. The reduction in section due to convergence of the radii has been expressed by a fraction, k, which is the ratio of the base length of the dam, b, to the radius of curvature of the up-stream face, R. If the section has a length across the up-stream face of 1 ft., the toe of the section has then a length across

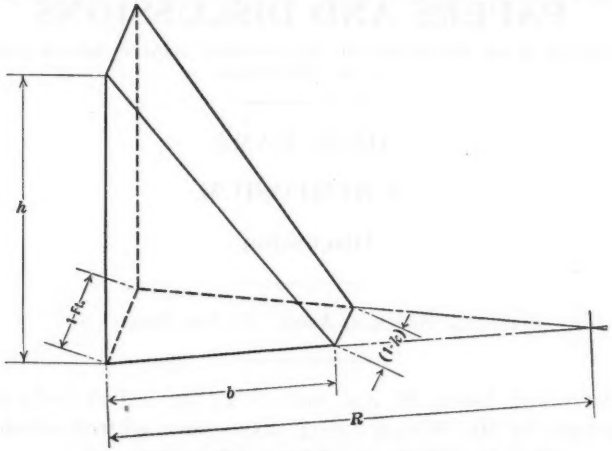

FIG. 6.—TRIANGULAR SECTION OF A GRAVITY DAM.

the river of $(1 - k)$ ft. In order to simplify the work, the water level was taken at the top of the section, and no uplift pressures were assumed. The base length was determined from the consideration that the resultant pressure with a straight dam, $k = 0$, should intersect the base at the down-stream third point. Thus,

$$b = h\sqrt{\frac{w}{\rho}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, w is the weight of water per cubic foot, and ρ is the weight of the concrete per cubic foot. Computations were then made to determine the stress intensities at the up-stream face of the dam for any value of k and, hence, for any radius of curvature. Without going into the details of the mechanics and algebra, the stress intensity, S, at the up-stream face, when considered as a vertical cantilever, can be found from Equation (2):

$$S = -\frac{\rho\, h\, k\, (6\,k^2 - 22\,k + 21)}{6\,(2 - k)\,(6 - 6\,k + k^2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The negative sign denotes a stress which is tensile for any degree of convergence as expressed through values of k between zero and 1. Let $\rho = 150$ and $h = 100$; then Equation (2) may be solved for S by introducing successive values of k, as follows:

k.	S, in pounds per square foot.
0	0 tension
0.1	492 "
0.2	967 "

As for the presence of arch action, the writer has estimated the deflection of the section shown in Fig. 6 when considered as a vertical cantilever on unyielding foundations. Let E, for concrete, = 3 000 000 lb. per sq. in., ρ = 150 lb. per cu. ft., and h = 100 ft.; then, the top of the section deflects down stream about 0.032 in. With such a small deflection the convergence of the radii would not close up any contraction joints such as are used ordinarily between blocks of the dam, and, therefore, no arch action can develop. Arch action can be expected only when the contraction joints between blocks are carefully grouted at a time of low temperature or when the arch has a comparatively short radius of curvature. Since these conditions are not often found, it may be said that arch action in a curved gravity dam cannot be expected.

Finally, the writer would disagree with Mr. Wiley's reasoning which leads him to conclude* that "there is no uplift in the body of a homogeneous mass of concrete". If a section of the dam above any given elevation be removed, in order to represent it as a "free body", according to the usual methods of mechanics, the plane of separation will cross voids as well as solid material. If the voids contained water under pressure, the severance of the structure requires that pressures equal to the water pressure be exerted upward where these voids existed. This is the uplift pressure. The opposite and downward pressures are exerted upon the section of the structure below the line of separation and not on the upper section. If the voids contain water under pressure, then there will be uplift forces tending to create tensile stresses in the solid surrounding the voids and thus in the section of severance. Mr. Wiley's observation† of the seepage of water from the body of the St. Francis Dam for weeks after the failure must be considered as further conclusive evidence that water under pressure does permeate the voids in concrete dams.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2323.

† *Loc. cit.*, p. 2322.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM AUSTIN BASSETT, M. Am. Soc. C. E.*

DIED MAY 16, 1929.

William Austin Bassett was born on September 29, 1876, at Roxbury, Mass. His New England ancestry dates back to 1621. His father was Isaac Austin Bassett and his mother, Annie (Tuson) Bassett.

Mr. Bassett attended the Lawrence Scientific School at Harvard University, from which he was graduated in 1901, with the degree of Civil Engineer. From August, 1901, to October, 1904, he was with the Pittsburgh Division of the Pennsylvania Railroad Company, in the Altoona, Pa., Office, in charge of construction of two main-line bridges, double-tracking the Southwest Branch, and miscellaneous yard and terminal work. From October, 1904, to June, 1906, he was with the Pittsburgh, Pa., Bureau of Filtration, engaged in the construction of a filtration plant, filtration galleries, and settling basins of reinforced concrete design. In June, 1906, he was appointed Assistant Professor of Civil Engineering at Carnegie Technical School, at Pittsburgh, and also was engaged in special work for the Department of Water Supply of the City of Pittsburgh.

From June to November, 1909, Professor Bassett was with the New York City Board of Water Supply as Assistant Engineer on the the construction of the Rondout Tunnel at High Falls, N. Y., and from November, 1909, to November, 1910, he was Designing Engineer in charge of the drafting-room at the Cambridge Office of the Boston Elevated Railroad Company, engaged in the construction of the Cambridge-Boston Subway. From November, 1910, to November, 1911, he was Engineer in charge of construction, costs, and cost investigations, at the Donora, Pa., Steel Works, of the American Steel and Wire Company. From November, 1911, to April, 1913, he served as Associate Editor of *The Engineering Record.*

From April, 1913 to December, 1926, Professor Bassett was Chief of the Division of Engineering and Public Works of the New York Bureau of Municipal Research and National Institute of Public Administration, engaged in miscellaneous professional and consultant service to municipalities, counties, and States in the United States and Canada. On December 1, 1926, he joined the Faculty of the Massachusetts Institute of Technology to organize a Division of Municipal and Industrial Research, of which he was Director until his death. Some of the accomplishments under his direction were a comprehensive survey of industrial conditions and possibilities in and about Providence, R. I., an industrial survey of Bangor, Me., and civic surveys of Meriden, Conn., and Norwood, Mass.

He served on the Hoover Committee for the Simplification of Building Codes and Practices, and on the Advisory Board for Highway Research of the

* Memoir prepared by William W. Brush, M. Am. Soc. C. E.

National Research Council. He was a Trustee of the Village of Scarsdale, N. Y., from 1922 to 1926, and a Supervisor of Westchester County, New York, from 1923 to 1926. He was the author of "Problems of Road Administration", published in 1917, and contributed to the technical press on engineering matters related to municipal and State problems.

Professor Bassett was intensely interested in establishing higher and more efficient standards in carrying on municipal and other Governmental activities, and devoted a great deal of his time to this phase of engineering work. His criticisms were always constructive and the methods he adopted, together with his pleasant personality, added greatly in securing the co-operation of those with whom he came in contact. His death at a comparatively early age is a distinct loss to the community.

He was a member of the Harvard Society of Engineers, the American Society for Municipal Improvements, and the Harvard Club.

His widow, Grace Loring Bassett, to whom he was married on March 30, 1908, and a son, John Austin Bassett, now a Senior at Harvard University, survive him.

Professor Bassett was elected a Member of the American Society of Civil Engineers on August 9, 1920.

FREDERIC LESLIE DUDLEY, M. Am. Soc. C. E.*

DIED MARCH 26, 1929.

Frederic Leslie Dudley was born in Templeton, Mass., on August 16, 1858. He attended the public schools of Templeton and studied Civil Engineering at the Worcester Polytechnic Institute, at Worcester, Mass., from which he was graduated in 1879.

After leaving the Institute, Mr. Dudley spent more than a year as Draftsman with Edward P. Adams, Landscape Architect and Civil Engineer, of Boston, Mass.

This was the period of railroad expansion in the West and in February, 1881, Mr. Dudley went to Kansas, where he was engaged in railroad construction for a number of years, first with the Engineering Corps of the Atchison, Topeka, and Santa Fé Railroad Company, and the St. Louis, Iron Mountain, and Southern Railway Company; and, later, as Division Engineer of the Burlington and Missouri River Railroad Company on construction work in Nebraska and Colorado.

In 1888, Mr. Dudley returned to the East and entered the employ of the Edge Moor (Del.) Bridge Company. In this new line of work he started as Draftsman, then became Checker, and, in two years, was in charge of design as a Squad Leader. He supervised the detailing of all classes of structures; bridges, however, became his chosen specialty. At this time he began his work on swing bridges which later developed into all classes of movable bridges, in which field he became well known. He contributed much valuable information of a permanent nature to this branch of the Engineering Profession.

* Memoir prepared by R. G. Manning, M. Am. Soc. C. E.

In 1901, the Edge Moor Bridge Company became a part of the American Company Bridge and in January, 1904, Mr. Dudley was transferred to the new Ambridge, Pa., Plant of this Company, in charge of the detailing of movable bridges. In 1905, he became Engineer of the Drawing Room for Railroad Bridges, including the design of machinery for movable bridges. For a number of years previous to his death his work also included the development of self-contained, power-driven railroad turn-tables and machinery for special movable structures, such as transfer-tables, ore and coal-handling bridges, etc. Until his retirement on January 1, 1929, just three months before his death, he continued in charge of this special work at the Ambridge Plant and as Consultant in the design of machinery for many movable structures.

During his twenty-five years at the Ambridge Plant, Mr. Dudley supervised the detailing of more than one hundred movable bridges of all types, including many of the larger swing and lift bridges, built by the American Bridge Company, in the United States. While in charge of the Drawing-Room for Railroad Bridges he supervised the detailing of many large structures, among which were the Mobile River Bridge, for the Louisville and Nashville Railroad Company (330-ft. draw span); the Hackensack River Bridge, for the Delaware, Lackawanna, and Western Railroad Company (a three-track, 198-ft. vertical lift bridge at Jersey City, N. J.); the Tombigbee River Bridge, for the Southern Railroad Company (184-ft. vertical lift span); the Rigolets Bridge, for the Louisville and Nashville Railroad Company (414-ft. draw span); the coal-handling bridge for the Carnegie Steel Company, at Clairton, Pa. (a double trolley, 400 ft. between shear leg and tower, 630 ft. over all, including the mechanical and electrical equipment).

The fixed long-span bridges on which Mr. Dudley was engaged, included a bridge for the Kentucky and Indiana Terminal Railroad Company, at Louisville, Ky. (two 615-ft. spans and one 400-ft. draw-span erected as cantilevers from adjoining spans, respectively, 275 and 373 ft.); the Ohio Connecting Railway Bridge, for the Pennsylvania Railroad Company, over the Ohio River, at Pittsburgh, Pa. (spans, 416 and 525 ft., respectively, each half of the latter erected as a cantilever using one-half of the former erected temporarily at each end as anchor arm); the suspension bridge over the Ohio River between Parkersburg, W. Va., and Belpre, Ohio (three spans, 775, 375, and 275 ft., and approaches); the 977½-ft. arch span for Hell Gate Bridge (six tracks) over the East River from Long Island to Ward's Island, N. Y., for the New York Connecting Railroad Company; and the single-track cantilever bridge over the Ohio River at Steubenville, Ohio (660-ft. channel span and 230-ft. anchor arms). Many of these bridges involved very special and difficult erection features, the details of which had to be developed in connection with the steel work.

While Mr. Dudley never entered the field as an author of technical books, he contributed valuable information to technical students, draftsmen, and engineers, in the form of a set of blue-print notes (54 pages) entitled, "Notes on Engineering Bridge Details." This pamphlet was widely distributed to college students and instructors in structural courses and to employees in the structural drawing-rooms all over the United States and Canada.

In the course of his practice, Mr. Dudley acquired a large amount of valuable information relating to the designing and detailing of machinery for movable bridges. This information, together with some added material, developed, through a committee of the American Bridge Company, composed of Messrs. Dudley, as Chairman, R. A. Mather, and P. J. Reich, M. Am. Soc. C. E., into a set of notes entitled, "Notes on Designing Machinery for Movable Bridges." These notes laid the foundation for an enlarged copy of nearly 200 pages developed under the supervision of O. E. Hovey, M. Am. Soc. C. E., Assistant Chief Engineer of the American Bridge Company. In his valuable two-volume work on "Movable Bridges," Mr. Hovey acknowledges the assistance and co-operation given by Mr. Dudley in its preparation.

In the death of Mr. Dudley, the Engineering Profession lost one of the most versatile men in the practical design and detail of movable bridges, inasmuch as the last thirty years of his life were intimately associated with the development of this class of structures, from the early types of the simple hand-operated swing bridge to the largest and most varied types of power-operated swing, bascule, and rolling bridges. He was also well versed in the detailing and designing of all types of steel structures and had an exceptionally keen insight into the fundamentals of engineering.

In acknowledging the worth of a member to the Society and the profession, one should not overlook his value to the community. Sterling in character, Mr. Dudley was of gentle and kindly disposition. He was a devoted husband and father and is survived by his widow, Clara Garvey Dudley, and a daughter, Ruth Dudley Daniells.

Mr. Dudley took an active part in the community life and devoted much time to the advancement of things worth while. He was a delightful companion, and much loved by his fellow citizens and co-workers.

Mr. Dudley was elected a Member of the American Society of Civil Engineers on October 10, 1916.

JAMES EDWARD FULTON, M. Am. Soc. C. E.*

DIED DECEMBER 6, 1928.

James Edward Fulton, the son of the late Hon. James Fulton, who represented the Taieri, New Zealand, Constituency in the House of Representatives many years prior to his elevation to the Legislative Council, was born at West Taieri, Otago, New Zealand, on December 11, 1854. His preliminary training was received in the District School at West Taieri which, together with intervals spent under private tutelage, completed his education.

When Mr. Fulton was still a very young man, he showed a special aptitude for engineering and in the early Seventies had sole charge of the machinery of a large flax mill for a period of nine months.

In January, 1874, he was articled to Mr. John Carruthers, of Wellington, who was then the New Zealand Government Engineer-in-Chief in the Colony in charge of railways. After Mr. Fulton had served for four years as a Cadet,

* Memoir prepared by M. Crompton-Smith, Esq., Wellington, New Zealand.

he passed the Junior and Senior Civil Service examinations, and also one required for the diploma of New Zealand Authorized and Licensed Surveyor. He was then appointed an Assistant Engineer of the Public Works Department, in principal charge of some railway location through heavily forested and rough country. He also served as First Assistant on the construction of 60 miles of railways, permanent way, and bridges, and had charge of the location and survey of 80 miles of additional railways.

In November, 1880, he resigned his position in the Government service to engage in private practice, but before doing so he was sent by the Department to the Bay of Islands, where he took soundings of the Harbor of Auckland and made a re-survey of part of the railway reserve lands there. During the period that Mr. Fulton was in private practice he went to Hawkes Bay to make some city re-surveys and, in 1882, to Auckland where he made a preliminary survey and estimate of the cost of the proposed line of the Kaihu Valley Railway Company.

From August, 1882, to 1897, Mr. Fulton was connected with the Wellington and Manawatu Railway Company, first as Resident Engineer under H. P. Higginson, Chief Engineer, in charge of the construction of the Palmerston-Waikanae Section. As Referee he also reviewed the various plans for the water supply and drainage of Palmerston North. In August, 1889, Mr. Fulton was promoted to the positions of Manager and Locomotive Superintendent to succeed his brother, the late A. R. W. Fulton. Mr. Fulton took a keen interest in his profession and was ever on the alert to give his Company the benefit of his inventions and improvements. It was while he was in the employ of the Wellington and Manawatu Railway Company that he invented and used in his office a telephone switchboard which was an attractive piece of mechanism, simple in manipulation, although of intricate appearance. He was associated with the movement to introduce from America the first compound locomotives that were used in New Zealand and that are still running with high efficiency on the Manawatu Railway.

Mr. Fulton retired from service with the Railway Company in 1897 and again entered private practice, undertaking the work of designing and building the Kelburne Cable Tramway in Wellington, which consisted of tunnels, viaducts, and retaining walls, amounting in value to more than £30 000. He also designed a great many bridges for County Councils and the New Zealand Government, and was the Designer and Engineer of a light railway for the Taupo-Totara Timber Company, 50 miles in length, which includes the longest wooden arch span (234 ft.) of any bridge in New Zealand. He served as Engineer for the construction of the railway for the Tongariro Timber Company.

Mr. Fulton's death occurred on December 6, 1928, after almost a year of failing health. As a man of high attainments in his profession and of estimable qualities in other respects, his passing was greatly regretted among a wide circle of friends, professional and otherwise, throughout the Dominion.

He was a member of the Institution of Civil Engineers of London, England, and of its Advisory Committee in New Zealand. He was also a member of the New Zealand Institute of Surveyors, of which he had served as President.

He is survived by his widow, Mrs. C. F. Fulton, of Wellington, and one daughter, who is the wife of Professor Horton, of Holloway College, in England.

Mr. Fulton was elected a Member of the American Society of Civil Engineers on October 4, 1910.

CHARLES FREDERICK TAYLOR, M. Am. Soc. C. E.*

DIED APRIL 14, 1927.

Charles Frederick Taylor was born on March 27, 1860, at Lewiston, Ill. He was but a little lad when his family moved to Mooers, N. Y., a small town in the Northern part of the State near the Canadian Border. Here he received his primary education and later, was prepared to enter Syracuse University, at Syracuse, N. Y.

Many amusing incidents are told of Mr. Taylor's early boyhood in the society of his several brothers and sisters. At the age of four, he had occasion to set forth on his first journey by train, going by way of the "Grand Trunk." He presented a very sad figure when through disobedience he lost his hat out of the car window and for penance was made to wear his sister's bonnet "home to grandpa's".

In 1880, Mr. Taylor and his brother, Henry L. Taylor, entered Syracuse University, where Mr. Taylor was enrolled for three and one-half years as a student in engineering. He was graduated in the Class of 1884 with the degree of Bachelor of Civil Engineering, and also with some months of special work at Rensselaer Polytechnic Institute, Troy, N. Y., to his credit. He received an honorary Civil Engineering Degree from his Alma Mater in 1905. He was affiliated with both the Phi Delta Theta and Sigma Psi Fraternities.

Mr. Taylor's active professional career included a wide variety of positions and great expanse of territory. The majority of these engagements are given in the following brief survey: From July to November, 1883, as Chainman and, afterward, Rodman and Leveler, on the construction of an extension of the Portland and Ogdensburg Railroad; from July to November, 1884, as Transitman on the construction of the Canada Atlantic Railroad; from May, 1885, to May, 1889, with Moffett, Hodgkins, and Clarke, on work including preliminary surveys of Grenell Island Park near the Thousand Islands and surveys at Oneida Lake for the Central City Water-Works Company of Syracuse; from September, 1885, to February, 1886, as Assistant Superintendent of Construction for the Newark, Ohio, Water-Works; from May, 1886, to January, 1887, as Construction Engineer on the Kankakee, Ill., Water-Works; from January to December, 1887, as Inspector of pipe laying and similar detail at the Canada Pipe Foundry for the Belleville, Ont., Canada, Water-Works; from April to December, 1888, as Construction Engineer for the Berlin, Ont., Water-Works and, later, with the Coburg, Ont., Water-Works. From May, 1889, to January, 1890, he served as Division Engineer on the construction of the Kinder-

* Memoir prepared from information supplied by Dr. Henry L. Taylor, University of the State of New York, Albany, N. Y., and Paul P. Wiant, M. Am. Soc. C. E.

hook and Hudson Railway from Niverville to Stuyvesant Falls, N. Y.; and after January, 1890, as Construction Engineer for the entire line as far as Hudson, N. Y.

In October, 1890, Mr. Taylor entered a field of service which gave him, personally, wide and rich experience in his chosen profession. He was engaged for six years with the Syracuse Water Board in charge of surveys at Skaneatles Lake and along the Skaneatles Outlet, and also on surveys connected with the condemnation proceedings to acquire the plant of the Water Company, water-power rights, etc. Early in his association with the Board he became Engineer in Charge of City Distribution of the Water-Works of the City of Syracuse. His faithful service in this employ put him in touch with financial backing that was in itself rare experience. During the last year of his service (1896), he personally supervised the laying of 80 miles of water mains.

From 1897 to 1903, Mr. Taylor acted in the capacity of Contractor for New York State on canal construction. During the latter part of this period, he also served as Treasurer of the Boston Tunnel Construction Company and probably the most noteworthy of his engineering feats was Section B on the East Boston Tunnel—more than a mile in length under the harbor. The tunnel was constructed "101 ft. below mean-tide level, the side walls and arch of concrete in the largest span ever constructed at that date in soft materials." It is remarkable that there occurred "the loss of but two human beings at a time when work was abandoned on the New York North River and the dead had not been removed from the flooded works." Also, at this time, the anthracite coal strike threatened alarming and continued financial disaster.

His Boston subway experience so fitted Mr. Taylor for under-water work, that he became Engineer or Treasurer of a dredging company operating in the Hudson Valley. The assembled plant made possible the successful bids for about one-third of the work of enlarging the New York State Canals under an appropriation of $9 000 000. In 1905, Mr. Taylor served as Treasurer and Chief Engineer of the Seely-Taylor Company, Engineers and Contractors, on a contract for the 30-in. submerged, cast-iron pipe sewer at New Rochelle, N. Y. At the conclusion of this work, he became President of the Maritime Dredging Company of New York, and was engaged on marine dredging work in New York Harbor until the close of the World War. His headquarters in New York demanded the presence of his family in the vicinity, and, after a few years in the city, he moved to New Rochelle where he resided until the death of his wife in the winter of 1918.

After a brief rest at Clifton Springs, N. Y., Mr. Taylor again took up his work, specializing in coal investigations, and, in this connection, he made extensive trips through the mining centers of the United States. In February, 1919, he began the settlement of his affairs and at a time of life when most men do not plan to undertake new and untried tasks, he offered himself to the Board of Foreign Missions of the Methodist Episcopal Church for service in any foreign field where he might be needed for building or other technical work. He was accepted early in 1922 for a short term of service in Foochow, China. The primary purpose of this engagement was to cover furloughs of the then

existing staff of the Fukien Construction Bureau, an organization for doing professional work in the Fukien Province of China.

Within a month after his arrival in Foochow, he was given full charge of an extensive building program, which he was able to manage very successfully, due to his extremely broad experience in the handling of materials and men. He constructed several notable buildings in this far distant country, which will stand for generations as a monument to his energy.

Mr. Taylor's term was to have been for three years, but this was extended to five, and would have been longer had it not been for local conditions which make all forms of building work dwindle to insignificant proportions during various intervals. He spent his month's vacation annually in travel. The first journey in 1923 in Northern China covered 3 000 miles; the second in 1924 through China, Korea, and Japan, 6 000 miles; the third in 1925 to Hongkong and The Philippines, 2 500 miles; and the fourth in 1926 to Chung King, West China, 4 000 miles.

When his work was completed in Foochow in December, 1926, he set out on his fifth and last journey, which was to be a leisurely trip around the world, with the intention of stopping at whatever place he might find something useful and worth while to do. He had traveled through the Straits Settlements, India, many of the East India Islands, Australia, and parts of South and Central Africa, when he was overtaken by illness at Old Umtali, in South Rhodesia. The end came very suddenly and peacefully, on August 14, 1927, while he was sitting in a chair reading a paper, apparently without the least premonition that death was near. His illness, which he thought to be only slight, proved to be myocarditis, and was fatal. A few days before he died, he had received a cable asking him to proceed to Monrovia, Liberia, to construct a school building for the Methodist Mission Board, which he was preparing to accept.

Mr. Taylor was a good business man and very honorable and upright in all his dealings. He was most painstaking and thorough, and never considered his work finished until it was done right. He had many of those qualities of geniality and brotherliness which made his friends think highly of him. By his outstanding character he made a very strong impression on the Chinese community in which he had lived for five years.

A letter from Mr. R. C. Gates, of Old Umtali, to Dr. Taylor, of Durban, South Africa, his nephew, bears the following message which tells of the affectionate memory in which Mr. Taylor is held:

"Our hearts were very heavy over the sudden death of our brother missionary who had so recently come among us, and who had so won our affection and esteem. Among other traits, I especially admired his high code of honor. But we all felt glad that since he he died on his journeys that he at least could die among friends and be buried on mission land where his grave can be looked after and his name kept in memory."

Mr. Taylor was married on March 27, 1888, to Emma R. Gibbs, of Hamilton, Ont., Canada, whose death preceded that of her husband. He is survived by two brothers, the Rev. B. S. Taylor, of Los Angeles, Calif., and Dr. Henry L. Taylor, of Albany, N. Y., and two sisters.

Mr. Taylor was elected an Associate Member of the American Society of Civil Engineers on September 4, 1895, and a Member on April 4, 1905.

WILLIAM HENRY WARREN, M. Am. Soc. C. E.*

DIED JANUARY 9, 1926.

William Henry Warren, whose life was so intimately associated with the development of engineering education in Australia, was born in Bristol, England, on February 2, 1852, in the year that also saw the birth of the American Society of Civil Engineers, an institution for whose work and publications he always expressed the most profound admiration. It was, indeed, one of Professor Warren's striking characteristics that, although in private life a somewhat vehemently patriotic Britisher, yet, in matters affecting the Engineering Profession, he was a whole-souled Internationalist. To his classes he always spoke enthusiastically of the achievements of American engineers and especially of the great bridge builders, some of whom were his warm friends. He maintained a correspondence not only with several of the leading engineers of his own land, but also with many in the different countries of Europe, especially with men engaged in his favorite study—the investigation of the properties of materials of construction. Thus, to his mind, such a happening as the dissolution of the International Association for Testing Materials following as an indirect consequence of the World War could seem nothing less than a tragedy.

William Henry Warren was one of the numerous company of young Englishmen of his generation, who felt that in the Colonies of their Motherland would be found a wider field and ampler scope for their energies and ambitions. His early training as a Civil Engineer was the fairly customary one of the Sixties in England. In those days there were, of course, practically no engineering colleges as understood to-day. A lad obtained the elements of his professional knowledge under articles of indenture to some engineering office or works. Young Warren left school soon after he was 14 years of age and entered on a six years' pupilage in the London and North Western Railway Works, at Crewe, England. During this period he passed through the various departments of this famous Railway Company under the direction, first, of Mr. J. Ramsbottom and afterward of Mr. F. W. Webb, both of whom were among the best-known of the British railway men of those days.

He next spent three years in attending ordinary collegiate courses of study in pure and applied science in London and in Manchester, England, and achieved a worthy list of distinctions in the various examinations. Among other honors he was awarded a Royal Exhibition, a Whitworth Scholarship, and the Society of Arts First Prize and Scholarship. Then, possessing a good foundation both of scientific knowledge and workshop practice, Mr. Warren devoted six years to accumulating experience in the design and construction of various works in the north of England, the most important of them being the large Municipal Gas Works at Manchester. This enterprise was, for its date, a very notable undertaking, and Mr. Warren, who acted in the capacity of Designing and Supervising Engineer for the contracting firm, always looked

* Memoir prepared by Sir Henry Barraclough, Dean of the Eng. Faculty, Univ. of Sydney, Sydney, N. S. W., Australia.

on it as his most valuable piece of training and as concluding the "journeyman" period of his life, after which he was ready to devote the remainder to whatever might happen to be the ultimate task to which he was called. He little thought, at the time, that this task was to be chiefly the education of students for the Engineering Profession.

It was in 1881 that Mr. Warren finally left England and took up what was to be his permanent residence in Sydney, New South Wales, Australia. Here, he immediately obtained a position in the Public Works Department of the State and, at the same time, undertook in the evenings the direction of the Department of Mechanical Engineering at the Sydney Technical College. Toward the end of 1883 the appointment of Lecturer-in-Charge of the newly established Department of Engineering at the University of Sydney was offered him, and he thus began the long period of University service which closed only a few days before his death.

In view of Professor Warren's retirement from the Russell School of Engineering on New Year's Day, 1926, the Governing Council of the University of Sydney formally adopted the following resolution, which conveniently summarizes his life's work and reputation:

"The Senate, having accepted with great regret, the resignation offered by Professor W. H. Warren, LL.D., of the Chair of Engineering, as from December 31st, 1925, hereby appoints him Emeritus Professor of Engineering from January 1st, 1926.

"At the date of his retirement Professor Warren will have completed 43 years in the service of the University, 42 of them as Professor of Engineering, this being the longest occupancy of any Professorial Chair in the history of the University since its foundation. Included in this period were terms of office as Chairman of the Professorial Board, as Dean of the Faculty of Science, and as the First Dean of the recently established Faculty of Engineering. Professor Warren has been associated with the Department of Engineering from its earliest beginnings to its present recognised position as one of the leading Schools of Engineering in the British Empire. His devotion to the task of building up sound systems of Engineering teaching and research in Australia, and his enthusiasm in helping adequately to develop the Engineering Profession, have made his name a household word amongst the Engineers of the Commonwealth.

"Particularly in connection with the testing and investigation of the materials of construction, Professor Warren has greatly enhanced the reputation of the University. By his unremitting labours in this direction for nearly forty years the University now possesses a magnificent laboratory for the testing of materials, and by his industry and energy in carrying out researches into our natural supplies of building materials, Professor Warren has added greatly to the industrial resources of the country.

"The Senate trusts that Professor Warren may live long to enjoy his leisure and to aid by his sympathy and advice the labours of those who follow after him."

This wish on the part of the Senate was not to be realized. Although arrangements had been made for Professor Warren to retain his old room at the University where he might do his technical writing—a plan that caused him much satisfaction—it was decreed that he should not return there again. The long vacation (of the Southern Hemisphere) had scarcely begun when the University was startled by his unexpected death. His end came suddenly

and without pain or distress on January 9, 1926, while he was talking to a friend in his own home at Elizabeth Bay, Sydney.

To those who knew him intimately there was something fitting, something almost characteristic, in his end. Having finished his work he turned promptly to the next thing, as was his customary way. Promptness was to Professor Warren a cardinal virtue. Procrastination and unpunctuality were no failings of his. As his colleagues often had occasion to notice, when a letter on his desk was opened and read, his hand instinctively reached toward a sheet of paper and a pen to begin the reply—he wrote almost all documents by hand at the University, rarely dictating anything. The only hopeful way of tempting him on occasion to delay dealing instantly with a paper was to offer to draft the reply for him; he cheerfully and gratefully admitted the seductiveness of this method when the subject-matter permitted it. This plan of allowing one or more of his colleagues to draft a document for his consideration, especially where it affected the Engineering Faculty at large, became one of the recognized methods which, in his later years, was adopted by general consent.

Within these few lines no detailed account is possible of Professor Warren's professional achievements, of his services to the community on Royal Commissions of inquiry, and on the Councils of scientific and technical Societies, nor of his work in the Engineering Department of the University of Sydney. From his colleagues and brother engineers his labors received ample recognition. For several years he represented Australia on the Council of the Institution of Civil Engineers (England); he was elected as the first President of the Institution of Engineers, Australia, and twice was President of the Royal Society of New South Wales; the distinction that gratified him more than any other was the honorary degree of Doctor of Laws conferred on him by the University of Glasgow.

He was the author of many technical papers and reports, but was probably best known by his two-volume textbook "Engineering Construction, Part 1, In Steel and Timber; Part 2, In Masonry and Concrete," which has obtained a wide circulation. His investigation work was confined almost entirely to the study of construction materials. The equipment of the University Laboratory for the testing of materials was one of his chief preoccupations. Even if at times he was thought by some to be spending too large a proportion of available funds on equipment for this Laboratory, his efforts resulted in getting together a magnificent collection of testing machines and measuring apparatus culled from the standard types of the different nations.

During his forty-three years of service Professor Warren had seen his Department grow from two rooms, as its quarters, and himself as the entire Teaching Staff, to a large Faculty of Professors and Lecturers, and a range of laboratories that have few equals in the Engineering Schools of the British Empire. These things gave him immense and undisguised pleasure. Every advance made by the School (against what, especially in the earlier and middle years, he regarded as neglect and opposition) caused him to exult greatly. The Engineering School of the University was his chief hobby as well as his task.

Perhaps, indeed, it is not inaccurate to state that Professor Warren's attitude, generally, to life was the amiable one of the "hobby-ist". He had definite limitations, or, perhaps, more accurately, he deliberately limited his activities to those things in which he was naturally and keenly interested, and he was somewhat impatient of the others. One can rarely remember seeing him at work on anything for which he did not seem to have a zest. It was the same in his spare hours; as all his friends well know, he then devoted himself with equal enthusiasm to his three hobbies—golf, prize bulldogs, and music, especially singing. His knowledge of operatic music was a surprise to many professional singers. His remarkably trained voice and the obvious enjoyment with which he would let himself be persuaded to sing a series of songs, will long be a pleasant tradition.

His most striking quality was a very human one—people felt there was something about him "you could not help liking." It was this personal quality, perhaps, that gave him many friends, even among men with whom he had strong differences of opinion and active controversy, and that made him so popular with the graduates and undergraduates of the Engineering School. The student who first called him "Bill," and the occasion for the remark, are not recorded, but they belong to the very early days of the School. The name gathered an atmosphere of affection and respect with each succeeding generation of students, and to be so addressed as he always was on festive occasions, gave the "Professor of Engineering" genuine satisfaction.

It is proverbially difficult to merit one's epitaph, but Professor Warren was not entirely unworthy of the motto of his family crest inscribed on a memorial tablet erected to his memory in an English church where he was once a choir boy—*Cadenti porrigo dextram.* Especially among his old students are there men who realize that his cordial right hand did them many a kind turn.

For the earlier graduates in Engineering, for many men scattered in engineering positions over the surface of the globe, as well as for those who, like the writer, have long been his colleagues, it is difficult to think of the Engineering School in Sydney apart from Professor Warren. We are conscious of an element of astonishment at the idea of the School without him. We find it hard to imagine laboratories and lecture rooms in which his familiar figure does not come and go. It was, with us, in the Engineering School as if

> "* * * having lived thus long, there seemed
> No need the King should ever die."

The Russell School of Engineering with which Professor Warren was so intimately associated was setting off afresh the year he left it. New buildings were coming, additional laboratories were in project, and a larger staff called for. The School's long future will, doubtless, be greater in achievement than its past, but no other period can equal in romantic interest its first forty years, and no single individual will ever be so uniquely identified with its development as its first and (as it turned out) its only "Professor of Engineering."

Professor Warren was elected a Member of the American Society of Civil Engineers on February 5, 1890.